Appendix E

Characteristics and Earthquake Performance of RVS Building Types

E.1 Introduction

For the purpose of the RVS, building structural framing types have been categorized into fifteen types listed in Section 3.7.1 and shown in Table 3-1. This appendix provides additional information about each of these structural types, including detailed descriptions of their characteristics, common types of earthquake damage, and common seismic rehabilitation techniques.

E.2 Wood Frame (W1, W2)

E.2.1 Characteristics

Wood frame structures are usually detached residential dwellings, small apartments, commercial buildings or one-story industrial structures. They are rarely more than three stories tall, although older buildings may be as high as six stories, in rare instances. (See Figures E-1 and E-2)

Figure E-1 Single family residence (an example of the W1 identifier, light wood-frame residential and commercial buildings less than 5000 square feet).

Wood stud walls are typically constructed of 2-inch by 4-inch wood members vertically set about 16 inches apart. (See Figures E-3 and E-4). These walls are braced by plywood or equivalent material, or by diagonals made of wood or steel. Many detached single family and low-rise multiple family residences in the United States are of stud wall wood frame construction.

Figure E-2 Larger wood-framed structure, typically with room-width spans (W2, light, wood-frame buildings greater than 5000 square feet).

Post and beam construction, which consists of larger rectangular (6 inch by 6 inch and larger) or sometimes round wood columns framed together with large wood beams or trusses, is not common and is found mostly in older buildings. These buildings usually are not residential, but are larger buildings such as warehouses, churches and theaters.

Timber pole buildings (Figures E-5 and E-6) are a less common form of construction found mostly in suburban and rural areas. Generally adequate seismically when first built, they are more often subject to wood deterioration due to the exposure of the columns, particularly near the ground surface. Together with an often-found “soft story” in this building type, this deterioration may contribute to unsatisfactory seismic performance.

In the western United States, it can be assumed that all single detached residential houses (i.e., houses with rear and sides separate from adjacent structures) are wood stud frame structures unless visual or supplemental information indicates otherwise (in the Southwestern U.S., for example, some residential homes are constructed of adobe, rammed earth, and other non-wood materials). Many houses that appear to have brick exterior facades are actually wood frame with nonstructural brick veneer or brick-patterned synthetic siding.

In the central and eastern United States, brick walls are usually not veneer. For these houses the

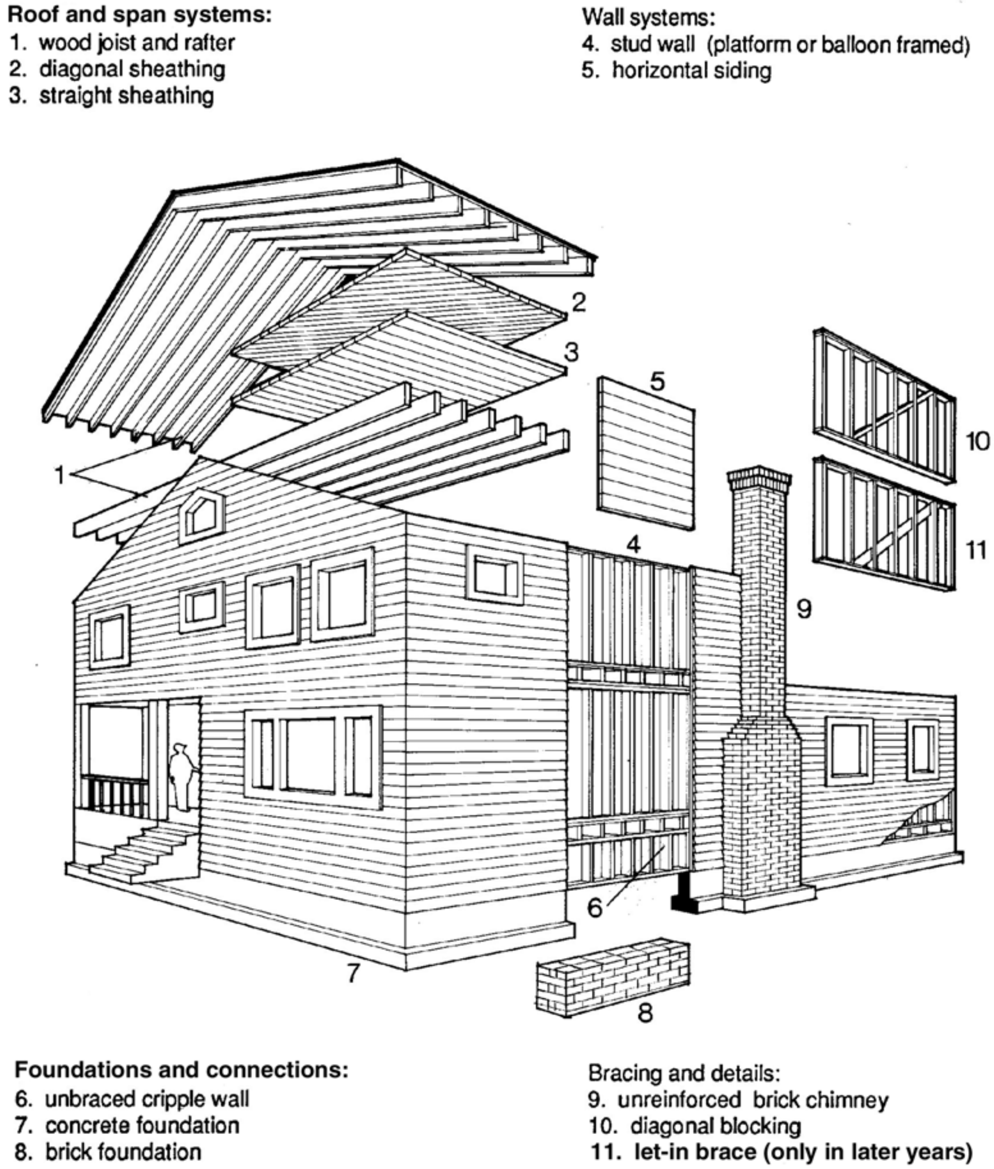


Figure E-3 Drawing of wood stud frame construction.

brick-work must be examined closely to verify that it is real brick. Second, the thickness of the exterior wall is estimated by looking at a window or door opening. If the wall is more than 9 inches from the interior finish to exterior surface, then it may be a brick wall. Third, if header bricks exist in the brick pattern, then it may be a brick wall. If these features all point to a brick wall, the house can be assumed to be a masonry building, and not a wood frame.

In wetter, humid climates it is common to find homes raised four feet or more above the outside grade with this space totally exposed (no foundation walls). This allows air flow under the house, to minimize decay and rot problems associated with high humidity and enclosed spaces. These houses are supported on wood post and small precast concrete pads or piers. A common name for this construction is post and pier construction.

E.2.2 Typical Earthquake Damage

Stud wall buildings have performed well in past earthquakes due to inherent qualities of the structural system and because they are lightweight and low-rise. Cracks in any plaster or stucco may appear, but these seldom degrade the strength of the building and are classified as nonstructural damage. In fact, this

Figure E-4 Stud wall, wood-framed house.

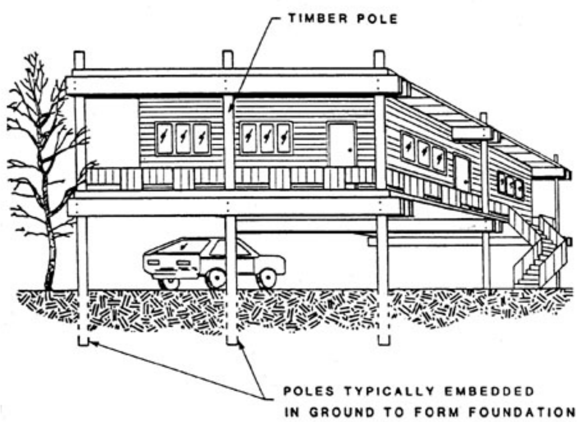


Figure E-5 Drawing of timber pole framed house.

Figure E-6 Timber pole framed house.

type of damage helps dissipate the earthquake-induced energy of the shaking house. The most common type of structural damage in older buildings results from a lack of adequate connection between the house and the foundation. Houses can slide off their foundations if they are not properly bolted to the foundations. This movement (see Figure E-7) results in major damage to the building as well as to plumbing and electrical connections. Overturning of

Figure E-7 House off its foundation, 1983 Coalinga earthquake.

the entire structure is usually not a problem because of the low-rise geometry. In many municipalities, modern codes require wood structures to be adequately bolted to their foundations. However, the year that this practice was adopted will differ from community to community and should be checked.

Many of the older wood stud frame buildings have no foundations or have weak foundations of unreinforced masonry or poorly reinforced concrete. These foundations have poor shear resistance to horizontal seismic forces and can fail.

Another problem in older buildings is the stability of cripple walls. Cripple walls are short stud walls between the foundation and the first floor level. Often these have no bracing neither in-plane nor out-of-plane and thus may collapse when subjected to horizontal earthquake loading. If the cripple walls collapse, the house will sustain considerable damage and may collapse. In some older homes, plywood sheathing nailed to the cripple studs may have been used to rehabilitate the cripple walls. However, if the sheathing is not nailed adequately to the studs and

Figure E-8 Failed cripple stud wall, 1992 Big Bear earthquake.

foundation sill plate, the cripple walls will still collapse (see Figure E-8).

Homes with post and pier perimeter foundations, which are constructed to provide adequate air flow under the structure to minimize the potential for decay, have little resistance to earthquake forces. When these buildings are subjected to strong earthquake ground motions, the posts may rotate or slip of the piers and the home will settle to the ground. As with collapsed cripple walls, this can be very expensive damage to repair and will result in the home building "red-tagged" per the ATC-20 post-earthquake safety evaluation procedures (ATC, 1989, 1995). See Figure E-9.

Figure E-9 Failure of post and pier foundation, Humboldt County.

Garages often have a large door opening in the front wall with little or no bracing in the remainder of the wall. This wall has almost no resistance to lateral forces, which is a problem if a heavy load such as a second story is built on top of the garage. Homes built over garages have sustained damage in past earthquakes, with many collapses. Therefore the house-over-garage configuration, which is found commonly in low-rise apartment complexes and some newer suburban detached dwellings, should be examined more carefully and perhaps rehabilitated.

Unreinforced masonry chimneys present a life-safety problem. They are often inadequately tied to the house, and therefore fall when strongly shaken. On the other hand, chimneys of reinforced masonry generally perform well.

Some wood-frame structures, especially older buildings in the eastern United States, have masonry veneers that may represent another hazard. The veneer usually consists of one wythe of brick (a wythe is a term denoting the width of one brick) attached to the stud wall. In older buildings, the veneer is either insufficiently attached or has poor quality mortar, which often results in peeling of the veneer during moderate and large earthquakes.

Post and beam buildings (not buildings with post and pier foundations) tend to perform well in earthquakes, if adequately braced. However, walls often do not have sufficient bracing to resist horizontal motion and thus they may deform excessively.

E.2.3 Common Rehabilitation Techniques

In recent years, especially as a result of the Northridge earthquake, emphasis has been placed on addressing the common problems associated with light-wood framing. This work has concentrated mainly in the western United States with single-family residences.

The rehabilitation techniques focus on houses with continuous perimeter foundations and cripple walls. The rehabilitation work consists of bolting the house to the foundation and providing plywood or other wood sheathing materials to the cripple walls to strengthen them (see Figure E-10). This is the most cost-effective rehabilitation work that can be done on a single-family residence.

Little work has been done in rehabilitating timber pole buildings or post and pier construction. In timber pole buildings rehabilitation techniques are focused on providing resistance to lateral forces by bracing (applying sheathing) to interior walls, creating a continuous load path to the ground. For homes with post and pier perimeter foundations, the work has focused on providing partial foundations and bracing to carry the earthquake loads.

Figure E-10 Seismic strengthening of a cripple wall, with plywood sheathing.

E.3 Steel Frames (S1, S2)

E.3.1 Characteristics

Steel frame buildings generally may be classified as either moment-resisting frames or braced frames, based on their lateral-force-resisting systems. Moment-resisting frames resist lateral loads and deformations by the bending stiffness of the beams and columns (there is no diagonal bracing). In concentric braced frames the diagonal braces are connected, at each end, to the joints where beams and columns meet. The lateral forces or loads are resisted by the tensile and compressive strength of the bracing. In eccentric braced frames, the bracing is slightly offset from the main beam-to-column connections, and the short section of beam is expected to deform significantly in bending under major seismic forces, thereby dissipating a considerable portion of the energy of the vibrating building. Each type of steel frame is discussed below.

Moment-Resisting Steel Frame

Typical steel moment-resisting frame structures usually have similar bay widths in both the transverse and longitudinal direction, around 20-30 ft (Figure E-11). The load-bearing frame consists of beams and columns distributed throughout the building. The floor diaphragms are usually concrete,

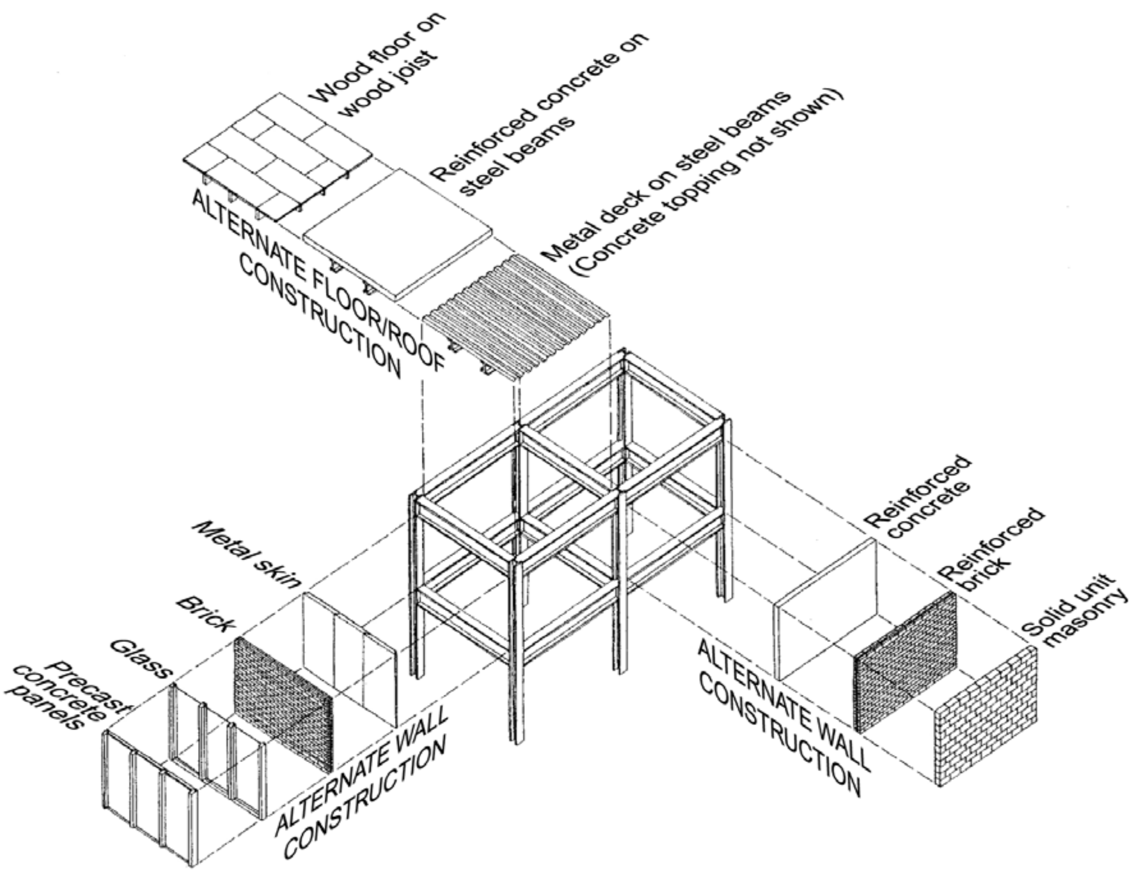


Figure E-11 Drawing of steel moment-resisting frame building.

sometimes over steel decking. Moment-resisting frame structures built since 1950 often incorporate prefabricated panels hung onto the structural frame as the exterior finish. These panels may be precast concrete, stone or masonry veneer, metal, glass or plastic.

This structural type is used for commercial, institutional and other public buildings. It is seldom used for low-rise residential buildings.

Steel frame structures built before 1945 are usually clad or infilled with unreinforced masonry such as bricks, hollow clay tiles and terra cotta tiles and therefore should be classified as S5 structures (see Section E.6 for a detailed discussion). Other frame buildings of this period are encased in concrete. Wood or concrete floor diaphragms are common for these older buildings.

Braced Steel Frame

Braced steel frame structures (Figures E-12 and E-13) have been built since the late 1800s with similar usage and exterior finish as the steel moment-frame buildings. Braced frames are sometimes used for long and narrow buildings because of their stiffness. Although these buildings are braced with diagonal members, the bracing members usually cannot be detected from the building exterior.

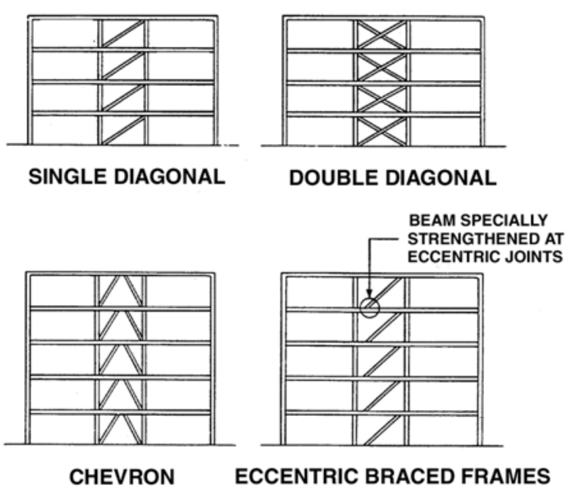


Figure E-12 Braced frame configurations.

From the building exterior, it is usually difficult to tell the difference between steel moment frames, braced frames, and frames with shear walls. In most modern buildings, the bracing or shear walls are located in the interior or covered by cladding material. Figure E-14 shows heavy diagonal bracing for a high rise building, located at the side walls, which

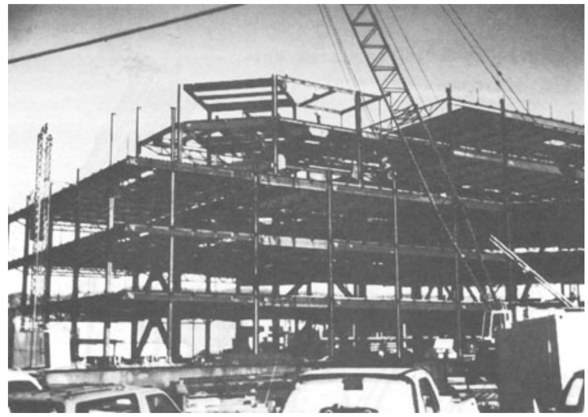


Figure E-13 Braced steel frame, with chevron and diagonal braces. The braces and steel frames are usually covered by finish material after the steel is erected.

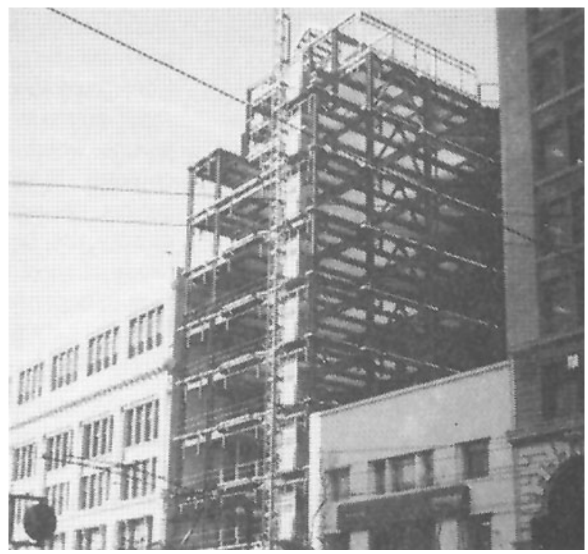


Figure E-14 Chevron bracing in steel building under construction.

will be subsequently covered by finish materials and will not be apparent. In fact, it is difficult to differentiate steel frame structures and concrete frame structures from the exterior. Most of the time, the structural members are clad in finish material. In older buildings, steel members can also be encased in concrete. There are no positive ways of distinguishing these various frame types except in the two cases listed below:

1. If a building can be determined to be a braced frame, it is probably a steel structure.

2. If exposed steel beams and columns can be seen, then the steel frame structure is apparent. (Especially in older structures, a structural frame which appears to be concrete may actually be a steel frame encased in concrete.)

E.3.2 Typical Earthquake Damage

Steel frame buildings tend to be generally satisfactory in their earthquake resistance, because of their strength, flexibility and lightness. Collapse in earthquakes has been very rare, although steel frame buildings did collapse, for example, in the 1985 Mexico City earthquake. In the United States, these buildings have performed well, and probably will not collapse unless subjected to sufficiently severe ground shaking. The 1994 Northridge and 1995 Kobe earthquakes showed that steel frame buildings (in particular S1 moment-frame) were vulnerable to severe earthquake damage. Though none of the damaged buildings collapsed, they were rendered unsafe until repaired. The damage took the form of broken welded connections between the beams and columns. Cracks in the welds began inside the welds where the beam flanges were welded to the column flanges. These cracks, in some cases, broke the welds or propagated into the column flange, "tearing" the flange. The damage was found in those buildings that experienced ground accelerations of approximately 20% of gravity (20%g) or greater. Since 1994 Northridge, many cities that experienced large earthquakes in the recent past have instituted an inspection program to determine if any steel frames were damaged. Since steel frames are usually covered with a finish material, it is difficult to find damage to the joints. The process requires removal of the finishes and removal of fireproofing just to see the joint.

Possible damage includes the following.

1. Nonstructural damage resulting from excessive deflections in frame structures can occur to elements such as interior partitions, equipment, and exterior cladding. Damage to nonstructural elements was the reason for the discovery of damage to moment frames as a result of the 1994 Northridge earthquake.
2. Cladding and exterior finish material can fall if insufficiently or incorrectly connected.
3. Plastic deformation of structural members can cause permanent displacements.
4. Pounding with adjacent structures can occur.

E.3.3 Common Rehabilitation Techniques

As a result of the 1994 Northridge earthquake many steel frame buildings, primarily steel moment frames, have been rehabilitated to address the problems discovered. The process is essentially to redo the connections, ensuring that cracks do not occur in the welds. There is careful inspection of the welding process and the electrodes during construction. Where possible, existing full penetration welds of the beams to the columns is changed so more fillet welding is

Figure E-15 Rehabilitation of a concrete parking structure using exterior X-braced steel frames.

used. This means that less heat is used in the welding process and consequently there is less potential for damage. Other methods include reducing welding to an absolute minimum by developing bolted connections or ensuring that the connection plates will yield (stretch permanently) before the welds will break. One other possibility for rehabilitating moment frames is to convert them to braced frames.

The kind of damage discovered was not limited to moment frames, although they were the most affected. Some braced frames were found to have damage to the brace connections, especially at lower levels.

Structural types other than steel frames are sometimes rehabilitated using steel frames, as shown for the concrete structure in Figure E-15. Probably the most common use of steel frames for rehabilitation is in unreinforced masonry bearing-wall buildings (URM). Steel frames are typically used at the storefront windows as there is no available horizontal resistance provided by the windows in their plane. Frames can be used throughout the first floor perimeter when the floor area needs to be open, as in a restaurant. See Figure E-16.

When a building is encountered with this type of rehabilitation scheme, the building should be considered a frame type building S1 or S2.

E.4 Light Metal (S3)

E.4.1 Characteristics

Most light metal buildings existing today were built after 1950 (Figure E-17).They are used for agricultural structures, industrial factories, and warehouses. They are typically one story in height, sometimes without interior columns, and often enclose a large floor area. Construction is typically of steel frames spanning the short dimension of the building, resisting lateral forces as moment frames. Forces in the long direction are usually resisted by diagonal steel rod bracing. These buildings are usually clad with lightweight metal or asbestos-reinforced concrete siding, often corrugated.

To identify this construction type, the screener should look for the following characteristics:

Figure E-16 Use of a braced frame to rehabilitate an unreinforced masonry building.

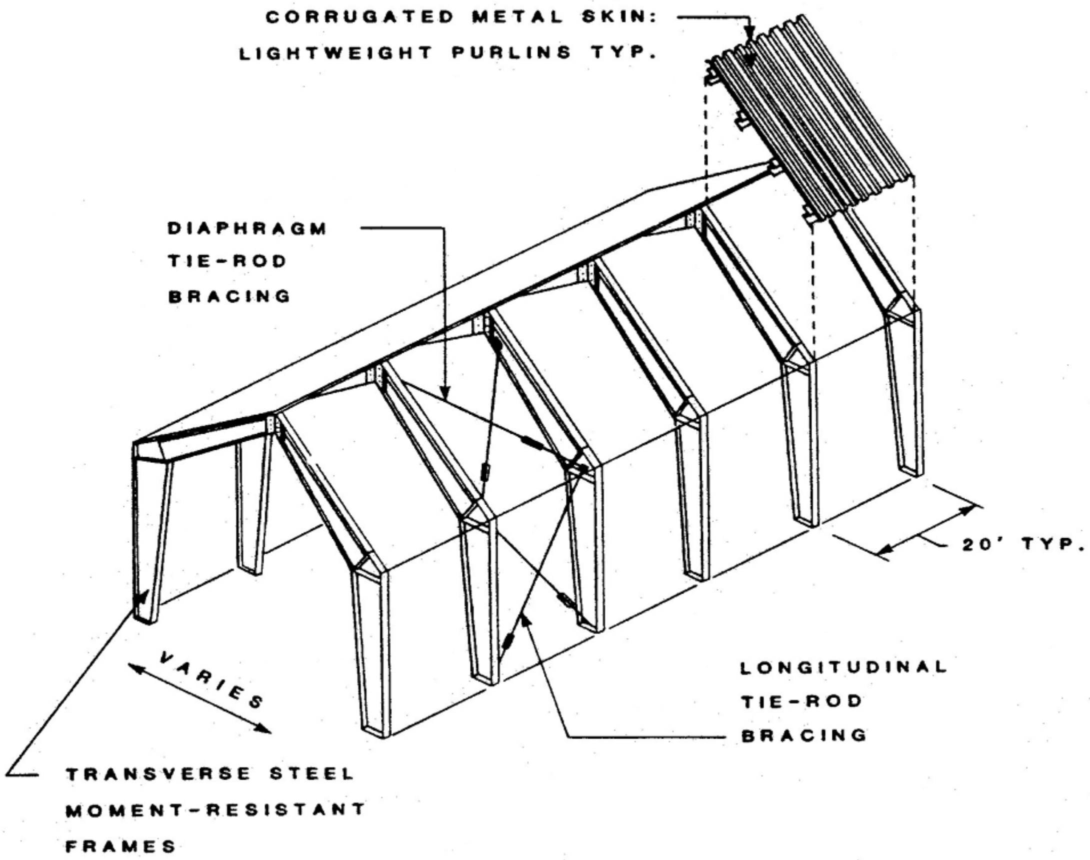


Figure E-17 Drawing of light metal construction.

1. Light metal buildings are typically characterized by industrial corrugated sheet metal or asbestos-reinforced cement siding. The term, "metal building panels" should not be confused with "corrugated sheet metal siding." The former are prefabricated cladding units usually used for large office buildings. Corrugated sheet metal siding is thin sheet material usually fastened to purlins, which in turn span between columns. If this sheet cladding is present, the screener should examine closely the fasteners used. If the heads of sheet metal screws can be seen in horizontal rows, the building is most likely a light metal structure (Figure E-18).

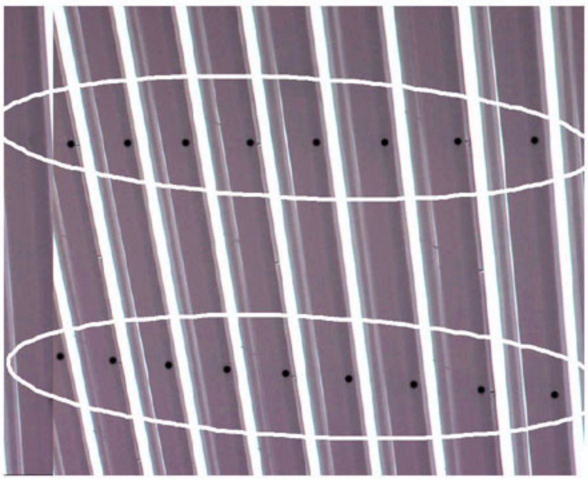


Figure E-18 Connection of metal siding to light metal frame with rows of screws (encircled).

2. Because the typical structural system consists of moment frames in the transverse direction and frames braced with diagonal steel rods in the longitudinal direction, light metal buildings often have low-pitched roofs without parapets or overhangs (Figure E-19). Most of these buildings are prefabricated, so the buildings tend to be rectangular in plan, without many corners.
3. These buildings generally have only a few windows, as it is difficult to detail a window in the sheet metal system.
4. The screener should look for signs of a metal building, and should knock on the siding to see if it sounds hollow. Door openings should be inspected for exposed steel members. If a gap, or light, can be seen where the siding meets the ground, it is certainly light metal or wood frame. For the best indication, an interior inspection will confirm the structural skeleton, because most of these buildings do not have interior finishes.

Figure E-19 Prefabricated metal building (S3, light metal building).

E.4.2 Typical Earthquake Damage

Because these building are low-rise, lightweight, and constructed of steel members, they usually perform relatively well in earthquakes. Collapses do not usually occur. Some typical problems are listed below:

1. Insufficient capacity of tension braces can lead to their elongation or failure, and, in turn, building damage.
2. Inadequate connection to the foundation can allow the building columns to slide.
3. Loss of the cladding can occur.

E.5 Steel Frame with Concrete Shear Wall (S4)

E.5.1 Characteristics

The construction of this structural type (Figure E-20) is similar to that of the steel moment-resisting frame in that a matrix of steel columns and girders is distributed throughout the structure. The joints, however, are not designed for moment resistance, and the lateral forces are resisted by concrete shear walls.

It is often difficult to differentiate visually between a steel frame with concrete shear walls and one without, because interior shear walls will often be covered by interior finishes and will look like interior nonstructural partitions. For the purposes of an RVS, unless the shear wall is identifiable from the exterior (i.e., a raw concrete finish was part of the architectural aesthetic of the building, and was left exposed), this building cannot be identified accurately. Figure E-21shows a structure with such an exposed shear wall. Figure E-22 is a close-up of shear wall damage.

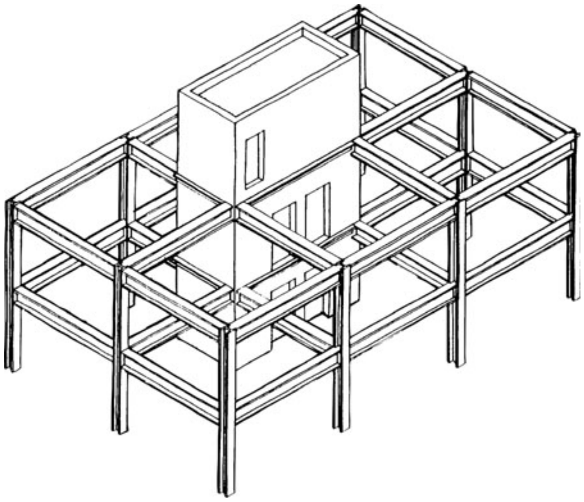


Figure E-20 Drawing of steel frame with interior concrete shear-walls.

Figure E-21 Concrete shear wall on building exterior.

E.5.2 Typical Earthquake Damage

The shear walls can be part of the elevator and service core, or part of the exterior or interior walls. This type of structure performs as well in earthquakes as other steel buildings. Some typical types of damage, other than nonstructural damage and pounding, are:

1. Shear cracking and distress can occur around openings in concrete shear walls.

Figure E-22 Close-up of exterior shear wall damage during a major earthquake.

2. Wall construction joints can be weak planes, resulting in wall shear failure at stresses below expected capacity.
3. Insufficient chord steel lap lengths can lead to wall bending failures.

E.6 Steel Frame with Unreinforced Masonry Infill (S5)

E.6.1 Characteristics

This construction type (Figures E-23 and E-24) consists of a steel structural frame and walls "infilled" with unreinforced masonry (URM). In older buildings, the floor diaphragms are often wood. Later buildings have reinforced concrete floors. Because of the masonry infill, the structure tends to be stiff. Because the steel frame in an older building is covered by unreinforced masonry for fire protection, it is easy to confuse this type of building with URM bearing-wall structures. Further, because the steel columns are relatively thin, they may be hidden in walls. An apparently solid masonry wall may enclose a series of steel columns and girders. These infill walls are usually two or three wythes thick. Therefore, header bricks will sometimes be present and thus mislead the screener into thinking the building is a URM bearing-wall structure, rather than infill. Often in these structures the infill and veneer masonry is exposed. Otherwise, masonry may be obscured by cladding in buildings, especially those that have undergone renovation.

When a masonry building is encountered, the screener should first attempt to determine if the masonry is reinforced, by checking the date of construction, although this is only a rough guide. A

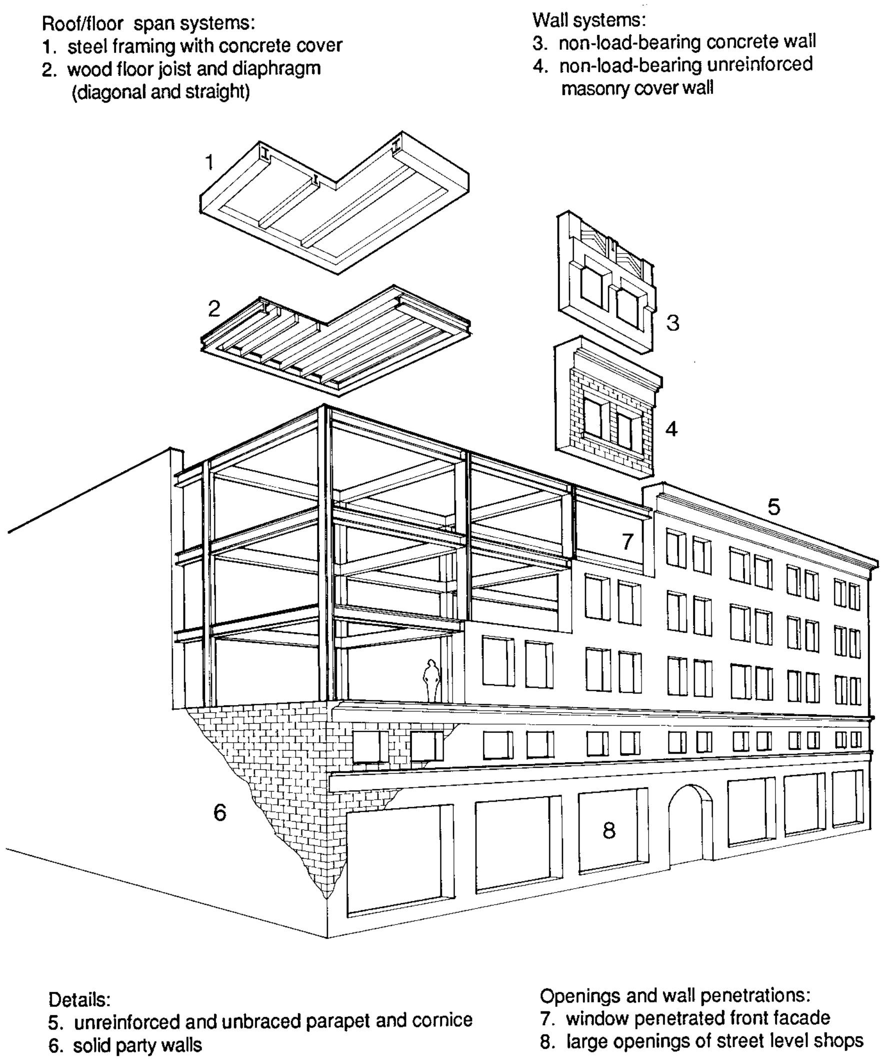


Figure E-23 Drawing of steel frame with URM infill.

clearer indication of a steel frame structure with URM infill is when the building exhibits the characteristics of a frame structure of type S1 or S2. One can assume all frame buildings clad in brick and constructed prior to about 1940 are of this type.

Older frame buildings may be of several types—steel frame encased with URM, steel frame encased with concrete, and concrete frame. Sometimes older buildings have decorative cladding such as terra cotta or stone veneer. Veneers may obscure all evidence of URM. In that case, the structural type cannot be determined. However, if there is evidence that a large amount of concrete is used in the building (for example, a rear wall constructed of concrete), then it is unlikely that the building has URM infill.

When the screener cannot be sure if the building is a frame or has bearing walls, two clues may help—the thickness of the walls and the height. Because infill walls are constructed of two or three wythes of bricks, they should be approximately 9 inches thick (2 wythes). Furthermore, the thickness of the wall will not increase in the lower stories, because the structural frame is carrying the load. For buildings over six stories tall, URM is infill or veneer, because URM bearing-wall structures are seldom this tall and, if so, they will have extremely thick walls in the lower stories.

E.6.2 Typical Earthquake Damage

In major earthquakes, the infill walls may suffer substantial cracking and deterioration from in-plane or out-of-plane deformation, thus reducing the in-plane wall stiffness. This in turn puts additional demand on the frame. Some of the walls may fail while others remain intact, which may result in torsion or soft story problems.

The hazard from falling masonry is significant as these buildings can be taller than 20 stories. As

Figure E-24 Example of steel frame with URM infill walls (S5).

described below, typical damage results from a variety of factors.

1. Infill walls tend to buckle and fall out-of-plane when subjected to strong lateral forces. Because infill walls are non-load-bearing, they tend to be thin (around 9") and cannot rely on the additional shear strength that accompanies vertical compressive loads.
2. Veneer masonry around columns or beams is usually poorly anchored to the structural members and can disengage and fall.
3. Interior infill partitions and other nonstructural elements can be severely damaged and collapse.
4. If stories above the first are infilled, but the first is not (a soft story), the difference in stiffness creates a large demand at the ground floor columns, causing structural damage.
5. When the earthquake forces are sufficiently high, the steel frame itself can fail locally. Connections between members are usually not designed for high lateral loads (except in tall buildings) and this can lead to damage of these connections. Complete collapse has seldom occurred, but cannot be ruled out.

E.6.3 Common Rehabilitation Techniques

Rehabilitation techniques for this structural type have focused on the expected damage. By far the most significant problem, and that which is addressed in most rehabilitation schemes, is failure of the infill wall out of its plane. This failure presents a significant life safety hazard to individuals on the exterior of the building, especially those who manage to exit the building during the earthquake. To remedy this problem, anchorage connections are developed to tie the masonry infill to the floors and roof of the structure.

Another significant problem is the inherent lack of shear strength throughout the building. Some of the rehabilitation techniques employed include the following.

1. Gunite (with pneumatically placed concrete) the interior faces of the masonry wall, creating reinforced concrete shear elements.
2. Rehabilitate the steel frames by providing cross bracing or by fully strengthening the connections to create moment frames. In this latter case, the frames are still not sufficient to resist all the lateral forces, and reliance on the infill walls is necessary to provide adequate strength.

For concrete moment frames the rehabilitation techniques have been to provide ductile detailing. This is usually done by removing the outside cover of concrete (a couple of inches) exposing the reinforcing ties. Additional ties are added with their ends embedded into the core of the column. The exterior concrete is then replaced. This process results in a detail that provides a reasonable amount of ductility but not as much as there would have been had the ductility been provided in the original design.

E.7 Concrete Moment-Resisting Frame (C1)

E.7.1 Characteristics

Concrete moment-resisting frame construction consists of concrete beams and columns that resist both lateral and vertical loads (see Figure E-25). A fundamental factor in the seismic performance of concrete moment-resisting frames is the presence or absence of ductile detailing. Hence, several construction subtypes fall under this category:

a. non-ductile reinforced-concrete frames with unreinforced infill walls,

b. non-ductile reinforced-concrete frames with reinforced infill walls,

c. non-ductile reinforced-concrete frames, and

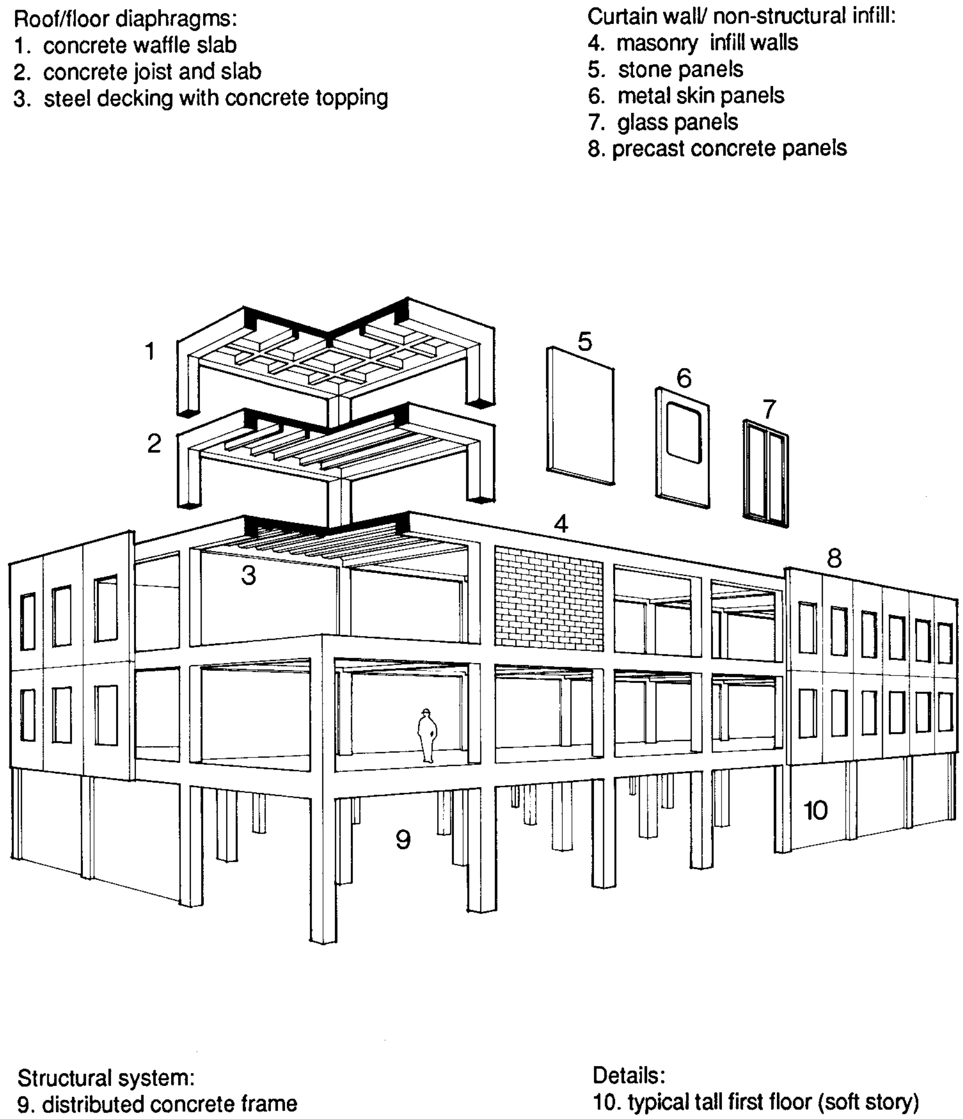


Figure E-25 Drawing of concrete moment-resisting frame building.

d. ductile reinforced-concrete frames.

Ductile detailing refers to the presence of special steel reinforcing within concrete beams and columns. The special reinforcement provides confinement of the concrete, permitting good performance in the members beyond the elastic capacity, primarily in bending. Due to this confinement, disintegration of the concrete is delayed, and the concrete retains its strength for more cycles of loading (i.e., the ductility is increased). See Figure E-26 for a dramatic example of ductility in concrete.

Ductile detailing (Figure E-27) has been practiced in high-seismicity areas since 1967, when ductility requirements were first introduced into the *Uniform Building Code* (the adoption and enforcement of ductility requirements in a given jurisdiction

Figure E-26 Extreme example of ductility in concrete, 1994 Northridge earthquake.

Figure E-27 Example of ductile reinforced concrete column, 1994 Northridge earthquake; horizontal ties would need to be closer for greater demands.

Figure E-28 Concrete moment-resisting frame building (C1) with exposed concrete, deep beams, wide columns (and with architectural window framing).

may be later, however). Prior to that time, nonductile or ordinary concrete moment-resisting frames were the norm (and still are, for moderate seismic areas). In high-seismicity areas additional tie reinforcing was required following the 1971 San Fernando earthquake and appeared in the *Uniform Building Code* in 1976.

In many low-seismicity areas of the United States, non-ductile concrete frames of type (a), (b), and (c) continue to be built. This group includes large multistory commercial, institutional, and residential buildings constructed using flat slab frames, waffle slab frames, and the standard beam-and-column frames. These structures generally are more massive than steel-frame buildings, are under-reinforced (i.e., have insufficient reinforcing steel embedded in the concrete) and display low ductility.

This building type is difficult to differentiate from steel moment-resisting frames unless the structural concrete has been left relatively exposed (see Figure E-28). Although a steel frame may be encased in concrete and appear to be a concrete frame, this is seldom the case for modern buildings (post 1940s). For the purpose of the RVS procedures, it can be assumed that all exposed concrete frames are concrete and not steel frames.

E.7.2 Typical Earthquake Damage

Under high amplitude cyclic loading, lack of confinement will result in rapid disintegration of nonductile concrete members, with ensuing brittle failure and possible building collapse (see Figure E-29).

Causes and types of damage include:

1. Excessive tie spacing in columns can lead to a lack of concrete confinement and shear failure.
2. Placement of inadequate rebar splices all at the same location in a column can lead to column failure.
3. Insufficient shear strength in columns can lead to shear failure prior to the full development of moment hinge capacity.
4. Insufficient shear tie anchorage can prevent the column from developing its full shear capacity.
5. Lack of continuous beam reinforcement can result in unexpected hinge formation during load reversal.

Figure E-29 Locations of failures at beam-to-column joints in nonductile frames, 1994 Northridge earthquake.

6. Inadequate reinforcing of beam-column joints or the positioning of beam bar splices at columns can lead to failures.
7. The relatively low stiffness of the frame can lead to substantial nonstructural damage.
8. Pounding damage with adjacent buildings can occur.

E.7.3 Common Rehabilitation Techniques

Rehabilitation techniques for reinforced concrete frame buildings depend on the extent to which the frame meets ductility requirements. The costs associated with the upgrading an existing, conventional beam-column framing system to meet the minimum standards for ductility are high and this approach is usually not cost-effective. The most practical and cost-effective solution is to add a system of shear walls or braced frames to provide the required seismic resistance (ATC, 1992).

E.8 Concrete Shear Wall (C2)

E.8.1 Characteristics

This category consists of buildings with a perimeter concrete bearing-wall structural system or frame structures with shear walls (Figure E-30). The structure, including the usual concrete floor diaphragms, is typically cast in place. Before the 1940s, bearing-wall systems were used in schools, churches, and industrial buildings. Concrete shear-wall buildings constructed since the early 1950s are institutional, commercial, and residential buildings, ranging from one to more than thirty stories. Frame buildings with shear walls tend to be commercial and industrial. A common example of the latter type is a warehouse with interior frames and perimeter concrete walls. Residential buildings of this type are often mid-rise towers. The shear walls in these newer buildings can be located along the perimeter, as interior partitions, or around the service core.

Frame structures with interior shear walls are difficult to identify positively. Where the building is clearly a box-like bearing-wall structure it is probably a shear-wall structure. Concrete shear wall buildings are usually cast in place. The screener should look for signs of cast-in-place concrete. In concrete bearing-wall structures, the wall thickness ranges from 6 to 10 inches and is thin in comparison to that of masonry bearing-wall structures.

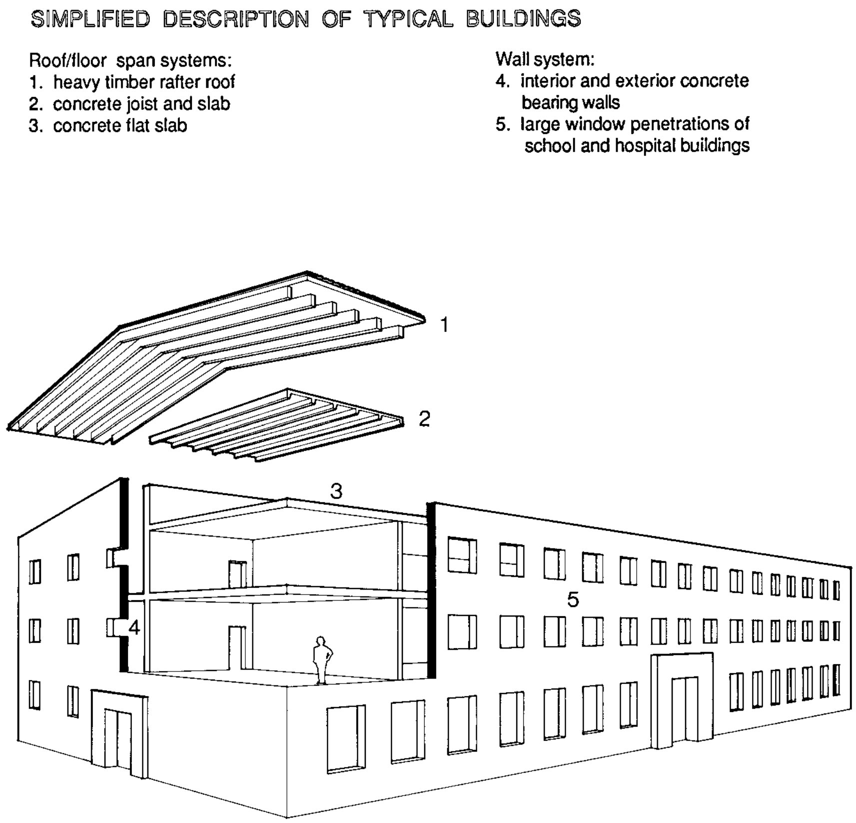


Figure E-30 Drawing of concrete shear-wall building.

E.8.2 Typical Types of Earthquake Damage

This building type generally performs better than concrete frame buildings. The buildings are heavy compared with steel frame buildings, but they are also stiff due to the presence of the shear walls. Damage commonly observed in taller buildings is caused by vertical discontinuities, pounding, and irregular configuration. Other damage specific to this building type includes the following.

1. During large seismic events, shear cracking and distress can occur around openings in concrete shear walls and in spandrel beams and link beams between shear walls (See Figures E-31 and E-32.)
2. Shear failure can occur at wall construction joints usually at a load level below the expected capacity.
3. Bending failures can result from insufficient vertical chord steel and insufficient lap lengths at the ends of the walls.

E.8.3 Common Rehabilitation

Reinforced concrete shear-wall buildings can be rehabilitated in a variety of ways. Techniques include: (1) reinforcing existing walls in shear by applying a layer of shotcrete or poured concrete; (2) where feasible, filling existing window or door openings with concrete to add shear strength and eliminate critical bending stresses at the edge of openings; and (3) reinforcing narrow overstressed shear panels in in-plane bending by adding reinforced boundary elements (ATC, 1992).

E.9 Concrete Frame with Unreinforced Masonry Infill (C3)

E.9.1 Characteristics

These buildings (Figures E-33 and E-34) have been, and continue to be, built in regions where unreinforced masonry (URM) has not been eliminated by code. These buildings were generally built before 1940 in high-seismicity regions and may continue to be built in other regions.

The first step in identification is to determine if the structure is old enough to contain URM. In contrast to steel frames with URM infill, concrete frames with URM infill usually show clear evidence of the concrete frames. This is particularly true for industrial buildings and can usually be observed at the side or rear of commercial buildings. The concrete col-

Figure E-31 Tall concrete shear-wall building: walls connected by damaged spandrel beams.

Figure E-32 Shear-wall damage, 1989 Loma Prieta earthquake.

umns and beams are relatively large and are usually not covered by masonry but left exposed.

A case in which URM infill cannot be readily identified is the commercial building with large windows on all sides; these buildings may have interior URM partitions. Another difficult case occurs when the exterior walls are covered by decorative tile or

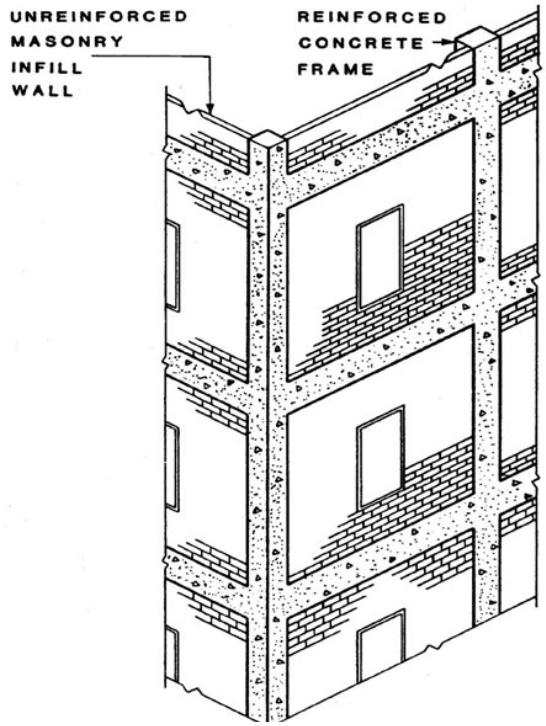


Figure E-33 Concrete frame with URM infill.

Figure E-34 Blow-up (lower photo) of distant view of C3 building (upper photo) showing concrete frame with URM infill (left wall), and face brick (right wall).

stone veneer. The infill material can be URM or a thin concrete infill.

E.9.2 Typical Earthquake Damage

The hazards of these buildings, which in the western United States are often older, are similar to and perhaps more severe than those of the newer concrete frames. Where URM infill is present, a falling hazard exists. The failure mechanisms of URM infill in a concrete frame are generally the same as URM infill in a steel frame.

E.9.3 Common Rehabilitation Techniques

Rehabilitation of unreinforced masonry infill in a concrete frame is identical to that of the URM infill in a steel frame. See Section E.6.3. Anchorage of the wall panels for out-of-plane forces is the key component, followed by providing sufficient shear strength in the building.

E.10 Tilt-up Structures (PC1)

E.10.1 Characteristics

In traditional tilt-up buildings (Figures E-35 through E-37), concrete wall panels are cast on the ground

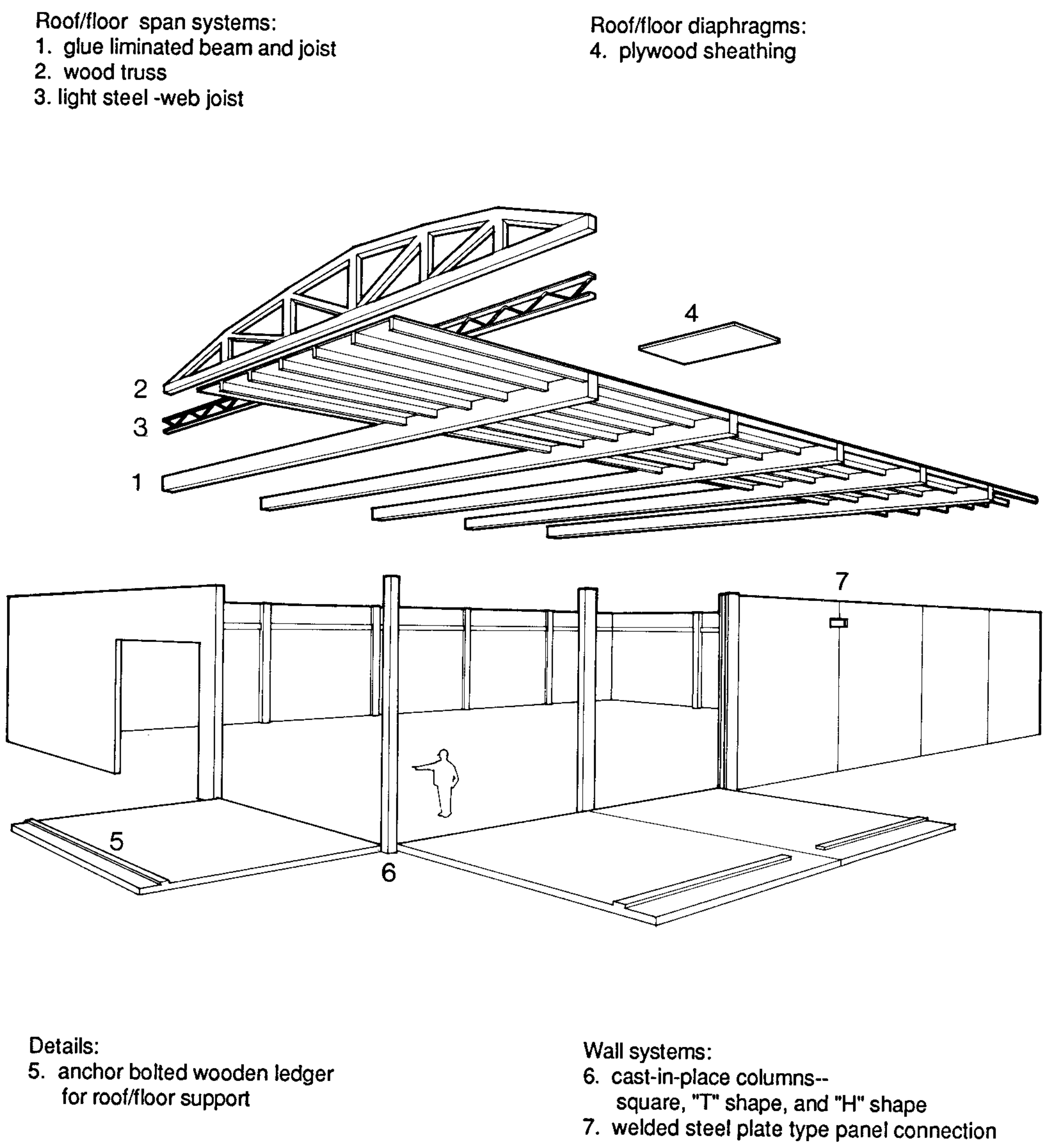


Figure E-35 Drawing of tilt-up construction typical of the western United States. Tilt-up construction in the eastern United States may incorporate a steel frame.

Figure E-36 Tilt-up industrial building, 1970s.

Figure E-37 Tilt-up industrial building, mid- to late 1980s.

and then tilted upward into their final positions. More recently, wall panels are fabricated off-site and trucked to the site.

Tilt-up buildings are an inexpensive form of light industrial and commercial construction and have become increasingly popular in the western and central United States since the 1940s. They are typically one and sometimes two stories high and basically have a simple rectangular plan. The walls are the lateral-force-resisting system. The roof can be a plywood diaphragm carried on wood purlins and glue-laminated (glulam) wood beams or a light steel deck and joist system, supported in the interior of the building on steel pipe columns. The wall panels are attached to concrete cast-in-place pilasters or to steel columns, or the joint is simply closed with a later concrete pour. These joints are typically spaced about 20 feet apart.

The major defect in existing tilt-ups is a lack of positive anchorage between wall and diaphragm, which has been corrected since about 1973 in the western United States.

In the western United States, it can be assumed that all one-story concrete industrial warehouses with flat roofs built after 1950 are tilt-ups unless supplementary information indicates otherwise.

E.10.2 Typical Earthquake Damage

Before 1973 in the western United States, many tilt-up buildings did not have sufficiently strong connections or anchors between the walls and the roof and floor diaphragms. The anchorage typically was nothing more than the nailing of the plywood roof sheathing to the wood ledgers supporting the framing.

During an earthquake, the weak anchorage broke the ledgers, resulting in the panels falling and the supported framing to collapse. When mechanical anchors were used they pulled out of the walls or split the wood members to which they were attached, causing the floors or roofs to collapse. See Figures E-38 and E-39. The connections between the concrete panels are also vulnerable to failure. Without these connections, the building loses much of its lateral-force-resisting capacity. For these reasons, many tilt-up buildings were damaged in the 1971 San

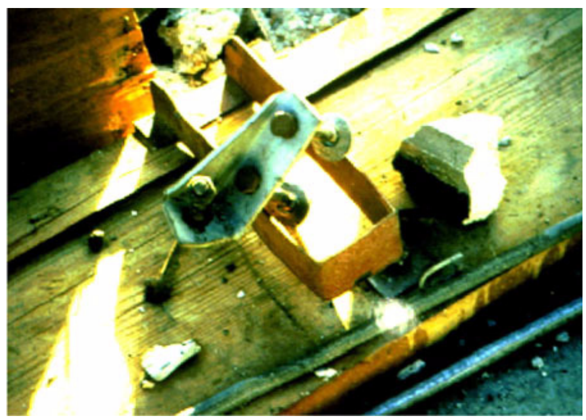


Figure E-38 Tilt-up construction anchorage failure.

Figure E-39 Result of failure of the roof beam anchorage to the wall in tilt-up building.

Fernando, California, earthquake.   Since 1973, tilt-up construction practices have changed in California and other high-seismicity regions, requiring positive wall-diaphragm connection. (Such requirements may not have yet been made in other regions of the country.) However, a large number of these older, pre-1970s-vintage tilt-up buildings still exist and have not been rehabilitated to correct this wall-anchor defect. Damage to these buildings was observed again in the 1987 Whittier, California, earthquake, 1989 Loma Prieta, California earthquake, and the 1994 Northridge, California, earthquake. These buildings are a prime source of seismic hazard.

In areas of low or moderate seismicity, inadequate wall anchor details continue to be used. Severe ground shaking in such an area may produce major damage in tilt-up buildings.

E.10.3 Common Rehabilitation Techniques

The rehabilitation of tilt-up buildings is relatively easy and inexpensive. The most common form of rehabilitation is to provide a positive anchorage connection at the roof and wall intersection. This is usually done by using pre-fabricated metal hardware attached to the framing member and to a bolt that is installed through the wall. On the outside of the wall a large washer plate is used. See Figure E-40 for examples of new anchors.

Accompanying the anchorage rehabilitation is the addition of ties across the building to develop the anchorage forces from the wall panels fully into the diaphragm. This is accomplished by interconnecting framing members from one side of the building to the other, and then increasing the connections of the diaphragm (usually wood) to develop the additional forces.

E.11 Precast Concrete Frame (PC2)

E.11.1 Characteristics

Precast concrete frame construction, first developed in the 1930s, was not widely used until the 1960s. The precast frame (Figure E-41) is essentially a post and beam system in concrete where columns, beams and slabs are prefabricated and assembled on site. Various types of members are used. Vertical-load-carrying elements may be Ts, cross shapes, or arches and are often more than one story in height. Beams are often Ts and double Ts, or rectangular sections. Prestressing of the members, including pretensioning and post-tensioning, is often employed. The identification of this structure type cannot rely solely on construction date, although most precast concrete

Figure E-40 Newly installed anchorage of roof beam to wall in tilt-up building.

frame structures were constructed after 1960. Some typical characteristics are the following.

1. Precast concrete, in general, is of a higher quality and precision compared to cast-in-place concrete. It is also available in a greater range of textures and finishes. Many newer concrete and steel buildings have precast concrete panels and column covers as an exterior finish (See Figure E-42). Thus, the presence of precast concrete does not necessarily mean that it is a precast concrete frame.
2. Precast concrete frames are, in essence, post and beam construction in concrete. Therefore, when a concrete structure displays the features of a post-and-beam system, it is most likely that it is a precast concrete frame. It is usually not economical for a conventional cast-in-place concrete frame to look like a post-and-beam system. Features of a precast concrete post-and-beam system include:

a. exposed ends of beams and girders that project beyond their supports or project away from the building surface,

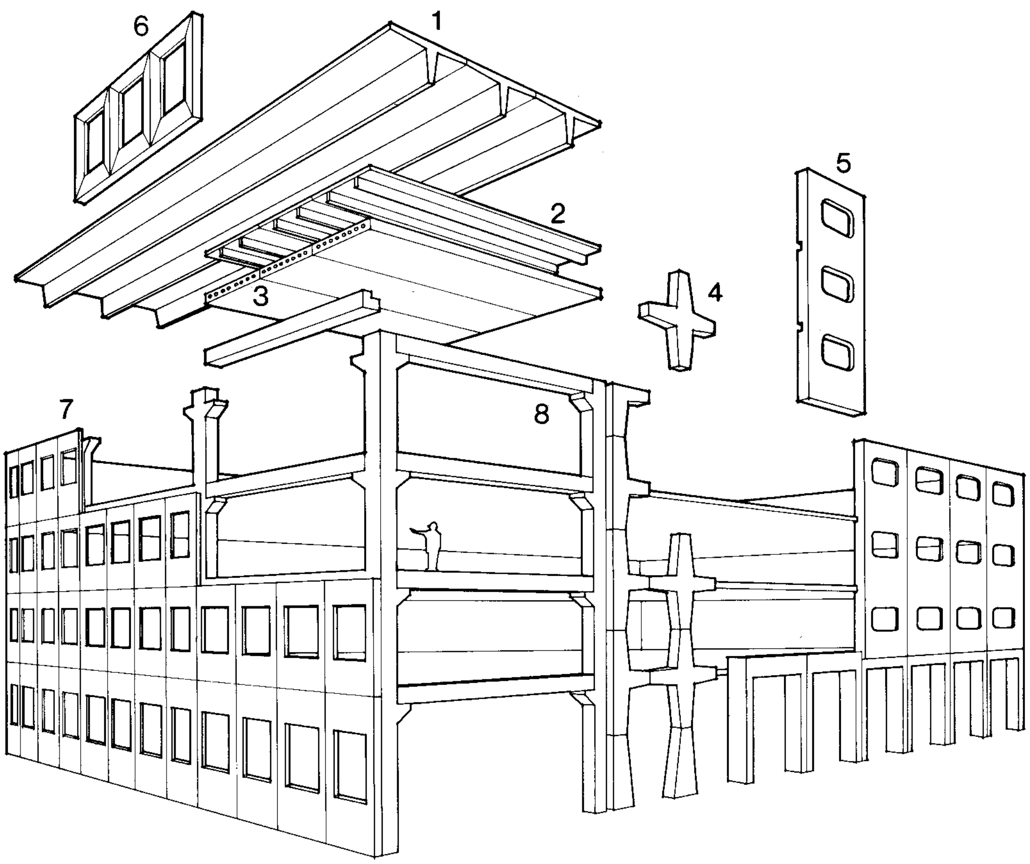


Figure E-41 Drawing of precast concrete frame building.

b. the absence of small joists, and

c. beams sitting on top of girders rather than meeting at a monolithic joint (see Figure E-43)

The presence of precast structural components is usually a good indication of this system, although these components are also used in mixed construction. Precast structural components come in a variety of shapes and sizes. The most common types are sometimes difficult to detect from the street. Less common but more obvious examples include the following.

a. Ts or double Ts—These are deep beams with thin webs and flanges and with large span capacities. (Figure E-44 shows one end of a double-T beam as it is lowered onto its seat.)

b. Cross or T-shaped units of partial columns and beams — These are structural units for constructing moment-resisting frames. They are usually joined together by field welding of steel connectors cast into the concrete. Joints should be clearly visible at the mid-span of the beams or the mid-height of the columns. See Figure E-45.

c. Precast arches—Precast arches and pedestals are popular in the architecture of these buildings.

d. Column—When a column displays a precast finish without an indication that it has a cover (i.e.,

Figure E-42 Typical precast column cover on a steel or concrete moment frame.

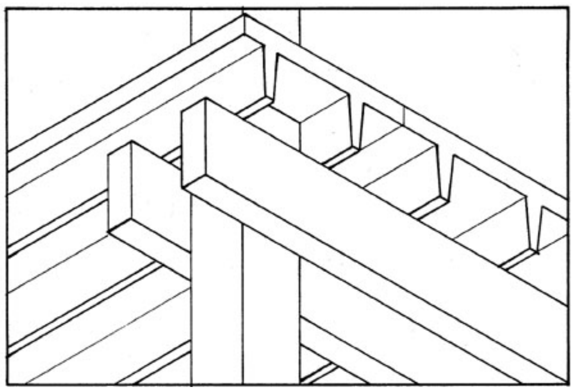


Figure E-43 Exposed precast double-T sections and overlapping beams are indicative of precast frames.

Figure E-44 Example of precast double-T section during installation.

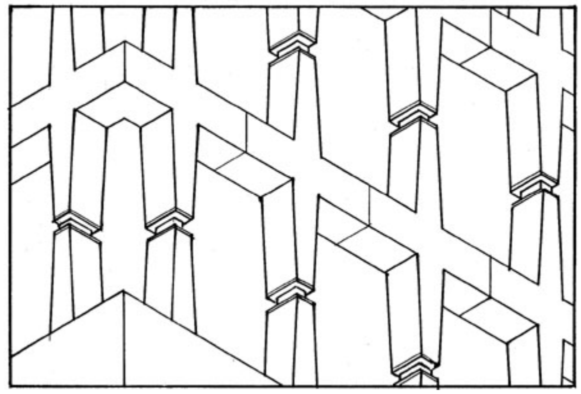


Figure E-45 Precast structural cross; installation joints are at sections where bending is minimum during high seismic demand.

no vertical seam can be found), the column is likely to be a precast structural column.

It is possible that a precast concrete frame may not show any of the above features, however.

E.11.2 Typical Earthquake Damage

The earthquake performance of this structural type varies widely and is sometimes poor. This type of building can perform well if the detailing used to connect the structural elements have sufficient strength and ductility (toughness). Because structures of this type often employ cast-in-place concrete or reinforced masonry (brick or block) shear walls for lateral-load resistance, they experience the same types of damage as other shear-wall building types. Some of the problem areas specific to precast frames are listed below.

1. Poorly designed connections between prefabricated elements can fail.
2. Accumulated stresses can result due to shrinkage and creep and due to stresses incurred in transportation.
3. Loss of vertical support can occur due to inadequate bearing area and insufficient connection between floor elements and columns.
4. Corrosion of the metal connectors between prefabricated elements can occur.

E.11.3 Common Rehabilitation Techniques

Seismic rehabilitation techniques for precast concrete frame buildings are varied, depending on the elements being strengthened. Inadequate shear capacity of floor diaphragms can be addressed by adding reinforced concrete topping to an untopped system when possible, or adding new shear walls to reduce the seismic shear forces in the diaphragm. Corbels with inadequate vertical shear or bending strength can be strengthened by adding epoxied horizontal shear dowels through the corbel and into the column. Alternatively, vertical shear capacity can be increased by adding a structural steel bolster under the corbel, bolted to the column, or a new steel column or reinforced concrete column can be added (ATC, 1992).

E.12 Reinforced Masonry (RM1 and RM2)

E.12.1 Characteristics

Reinforced masonry buildings are mostly low-rise structures with perimeter bearing walls, often with wood diaphragms (RM1 buildings) although precast concrete is sometimes used (RM2 buildings). Floor and roof assemblies usually consist of timber joists and beams, glued-laminated beams, or light steel joists. The bearing walls consist of grouted and reinforced hollow or solid masonry units. Interior supports, if any, are often wood or steel columns, wood stud frames, or masonry walls. Occupancy varies from small commercial buildings to residential and industrial buildings. Generally, they are less than five stories in height although many taller masonry buildings exist. Reinforced masonry structures are usually basically rectangular structures (See Figure E-46).

Figure E-46 Modern reinforced brick masonry.

To identify reinforced masonry, one must determine separately if the building is masonry and if it is reinforced. To obtain information on how to recognize a masonry structure, see Appendix D, which describes the characteristics of construction materials. The best way of assessing the reinforcement condition is to compare the date of construction with the date of code requirement for the reinforcement of masonry in the local jurisdiction.

The screener also needs to determine if the building is veneered with masonry or is a masonry building. Wood siding is seldom applied over masonry. If the front facade appears to be reinforced masonry whereas the side has wood siding, it is probably a wood frame that has undergone facade renovation. The back of the building should be checked for signs of the original construction type.

If it can be determined that the bearing walls are constructed of concrete blocks, they may be reinforced. Load-bearing structures using these blocks are probably reinforced if the local code required it. Concrete blocks come in a variety of sizes and textures. The most common size is 8 inches wide by 16 inches long by 8 inches high. Their presence is obvious if the concrete blocks are left as the finish surface.

E.12.2 Typical Earthquake Damage

Reinforced masonry buildings can perform well in moderate earthquakes if they are adequately reinforced and grouted, and if sufficient diaphragm anchorage exists. A major problem is control of the workmanship during construction. Poor construction practice can result in ungrouted and unreinforced walls. Even where construction practice is adequate, insufficient reinforcement in the design can be responsible for heavy damage of the walls. The lack of positive connection of the floor and roof diaphragms to the wall is also a problem.

E.12.3 Common Rehabilitation Techniques

Techniques for seismic rehabilitation of reinforced masonry bearing wall buildings are varied, depending on the element being rehabilitated. Techniques for rehabilitating masonry walls include: (1) applying a layer of concrete or shotcrete to the existing walls; (2) adding vertical reinforcing and grouting into ungrouted block walls; and (3) filling in large or critical openings with reinforced concrete or masonry dowelled to the surrounding wall. Wood or steel deck diaphragms in RM1 buildings can be rehabilitated by adding an additional layer of plywood to strengthen and stiffen an existing wood diaphragm, by shear welding between sections of an existing steel deck or adding flat sheet steel reinforcement, or by adding additional vertical elements (for example, shear walls or braced frames) to decrease diaphragm spans and stresses. Precast floor diaphragms in RM2 buildings can be strengthen by adding a layer of concrete topping reinforced with mesh (if the supporting structure has the capacity to carry the additional vertical dead load), or by adding new shear walls to reduce the diaphragm span (ATC, 1992).

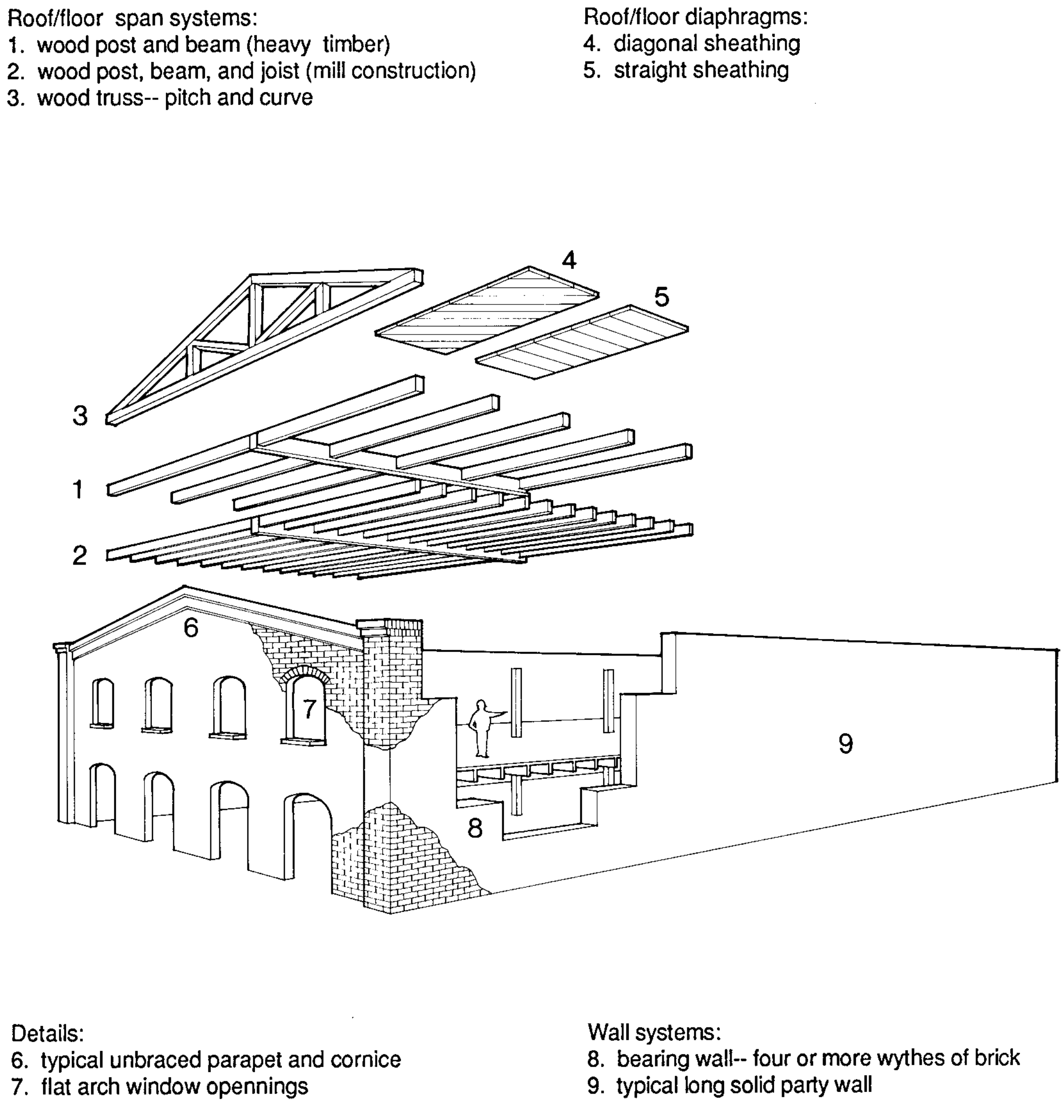


Figure E-47 Drawing of unreinforced masonry bearing-wall building, 2-story.

E.13 Unreinforced Masonry (URM)

E.13.1 Characteristics

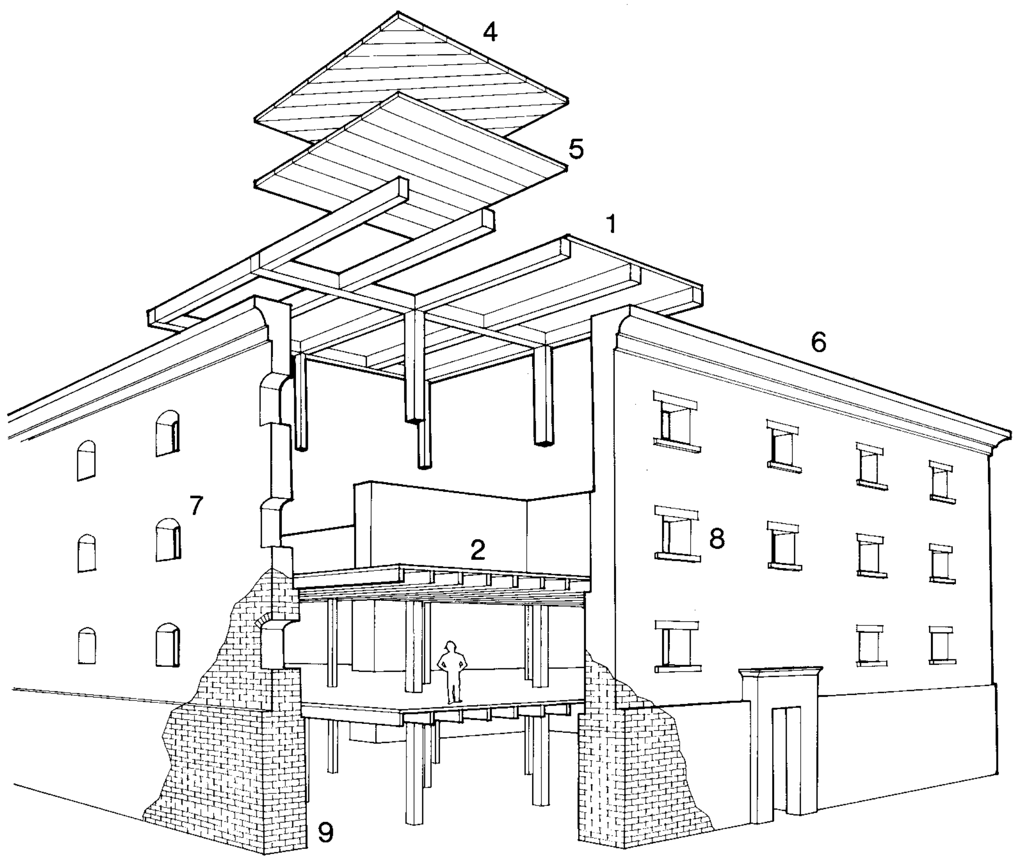
Most unreinforced masonry (URM) bearing-wall structures in the western United States (Figures E-47 through E-51) were built before 1934, although this construction type was permitted in some jurisdictions having moderate or high seismicity until the late 1940s or early 1950s (in some jurisdictions URM may still be a common type of construction, even today). These buildings usually range from one to six stories in height and function as commercial, residential, or industrial buildings. The construction varies according to the type of use, although wood floor and roof diaphragms are common. Smaller commercial and residential buildings usually have light wood floor joists and roof joists supported on the typical perimeter URM wall and interior, wood, load-bearing partitions. Larger buildings, such as industrial warehouses, have heavier floors and interior columns, usually of wood. The bearing walls of these industrial buildings tend to be thick, often as much as 24 inches or more at the base. Wall thickness of residential, commercial, and office buildings range from 9 inches at upper floors to 18 inches a lower floors.

The first step in identifying buildings of this type is to determine if the structure has bearing walls. Second, the screener should determine the approximate age of the building. Some indications of unreinforced masonry are listed below.

1. Weak mortar was used to bond the masonry units together in much of the early unreinforced

Roof/floor span systems:
1. wood post and beam (heavy timber)
2. wood post, beam, and joist (mill construction)
3. wood truss-- pitch and curve

Roof/floor diaphragms:
4. diagonal sheathing
5. straight sheathing



Details:
6. typical unbraced parapet and cornice
7. flat arch window opennings
8. small window penetrations (if bldg is originally a warehouse)

Wall systems:
9. bearing wall-- four to eight wythes of brick

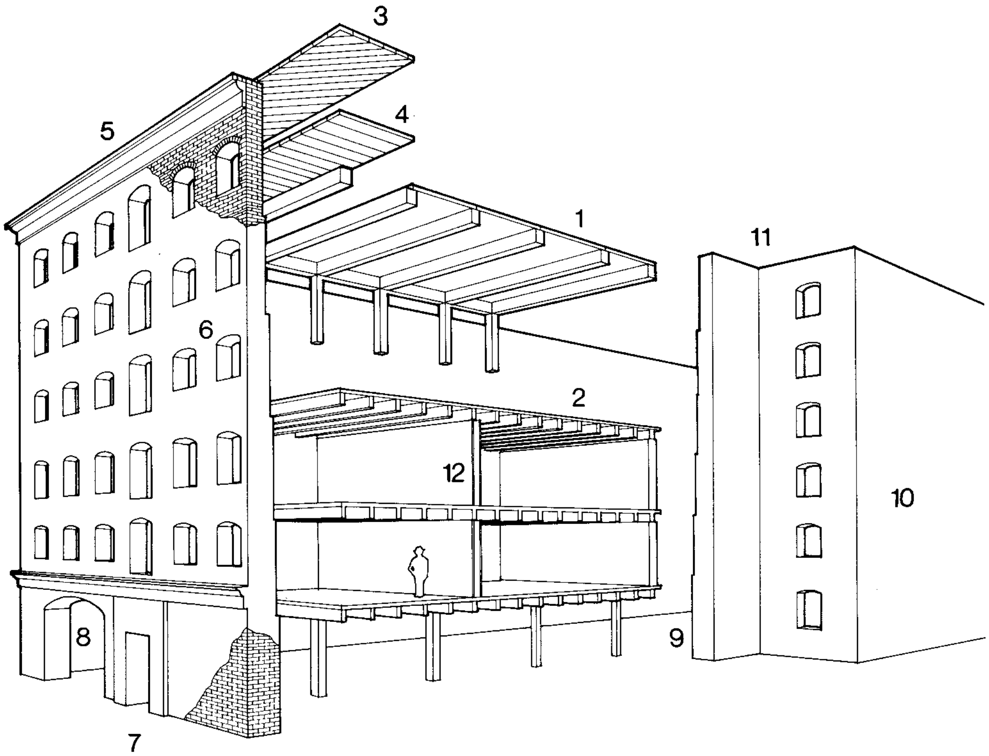
Figure E-48 Drawing of unreinforced masonry bearing-wall building, 4-story.

masonry construction in the United States. As the poor earthquake performance of this mortar type became known in the 1930s, and as cement mortar became available, this weaker mortar was not used and thus is not found in more recent masonry buildings. If this soft mortar is present, it is probably URM. Soft mortar can be scratched with a hard instrument such as a penknife, screwdriver, or a coin. This scratch testing, if permitted, should be done in a wall area where the original structural material is exposed, such as the sides or back of a building. Newer masonry may be used in renovations and it may look very much like the old. Older mortar joints can also be repointed (i.e., regular maintenance of the masonry mortar), or repaired with newer mortar during renovation. The original construction may also have used a high-quality mortar. Thus, even if the existence of soft mortar cannot be detected, it may still be URM.

2. An architectural characteristic of older brick bearing-wall structures is the arch and flat arch

Roof/floor span systems:
1. wood post and beam (heavy timber)
2. wood post, beam, and joist (mill construction)

Roof/floor diaphragms:
3. diagonal sheathing
4. straight sheathing



Details:
5. typical unbraced parapet and cornice
6. flat arch window opennings
7. typical penetrated facade of residential buildings
8. large opennings of ground floor shops

Wall systems:
9. bearing wall-- four to eight wythes of brick
10. typical long solid party wall
11. light/ventilation wells in residential bldg
12. non-structural wood stud partition walls

Figure E-49 Drawing of unreinforced masonry bearing-wall building, 6-story.

Figure E-50 East coast URM bearing-wall building.

Figure E-51 West coast URM bearing-wall building.

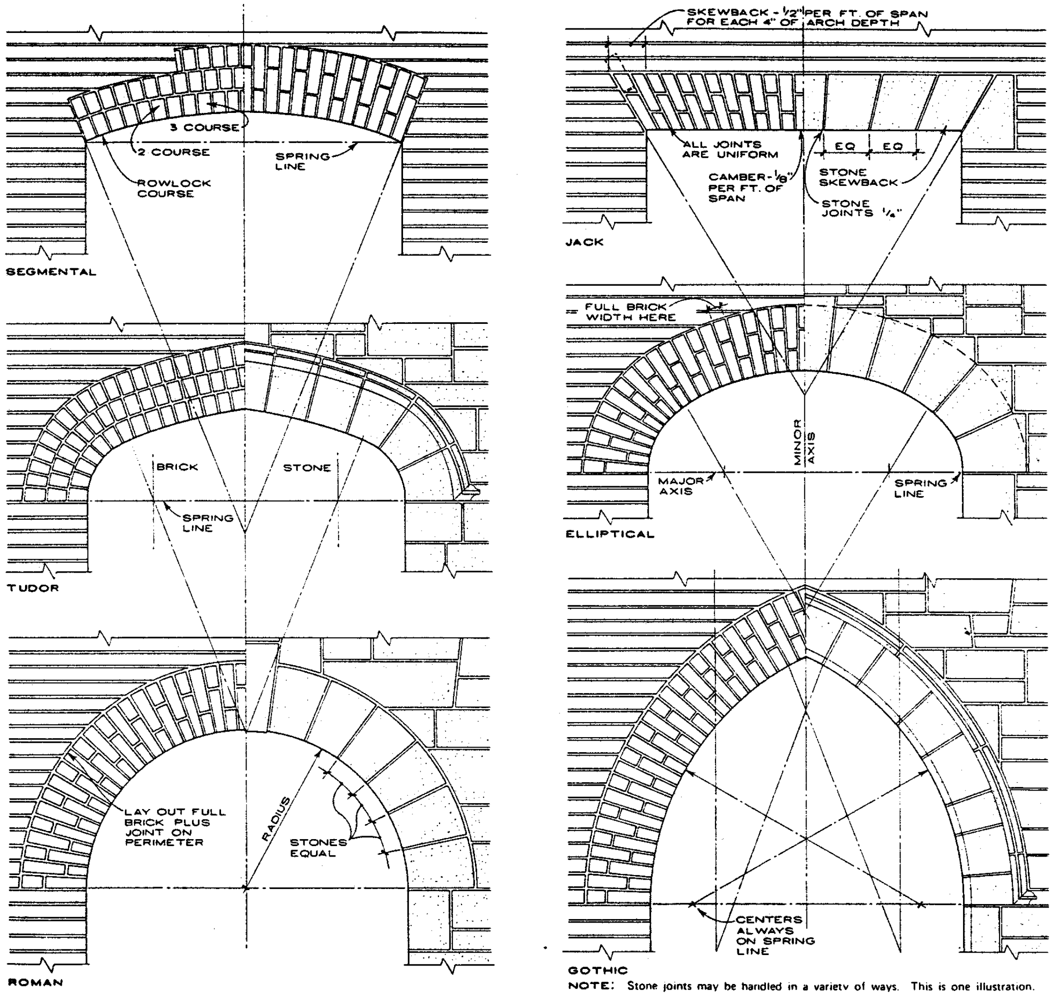


Figure E-52 Drawings of typical window head features in URM bearing-wall buildings.

window heads (see Figure E-52). These arrangements of masonry units function as a header to carry the load above the opening to either side. Although masonry-veneered wood-frame structures may have these features, they are much more widely used in URM bearing-wall structures, as they were the most economical method of spanning over a window opening at the time of construction. Other methods of spanning are also used, including steel and stone lintels, but these methods are generally more costly and usually employed in the front facade only.

3. Some structures of this type will have anchor plates visible at the floor and roof lines, approximately 6-10 feet on center around the perimeter of the building. Anchor plates are usually square or diamond-shaped steel plates approximately 6 inches by 6 inches, with a bolt and nut at the center. Their presence indicates anchor ties have been placed to tie the walls to the floors and roof. These are either from the original construction or from rehabilitation under local ordinances. Unless the anchors are 6 feet on center or less, they are not considered effective in earthquakes. If they are closely spaced, and appear to be recently installed, it indicates that the building has been rehabilitated. In either case, when these anchors are present all around the building, the original construction is URM bearing wall.

4. When a building has many exterior solid walls constructed from hollow clay tile, and no columns of another material can be detected, it is probably not a URM bearing wall but probably a wood or metal frame structure with URM infill.

5. One way to distinguish a reinforced masonry building from an unreinforced masonry building is to examine the brick pattern closely. Reinforced masonry usually does not show header bricks in the wall surface.

If a building does not display the above features, or if the exterior is covered by other finish material, the building may still be URM.

E.13.2 Typical Earthquake Damage

Unreinforced masonry structures are recognized as the most hazardous structural type. They have been observed to fail in many modes during past earthquakes. Typical problems include the following.

1. Insufficient Anchorage—Because the walls, parapets, and cornices are not positively anchored to the floors, they tend to fall out. The collapse of bearing walls can lead to major building collapses. Some of these buildings have anchors as a part of the original construction or as a rehabilitation. These older anchors exhibit questionable performance. (See Figure E-53 for parapet damage.)

Figure E-53 Parapet failure leaving an uneven roof line, due to inadequate anchorage, 1989 Loma Prieta earthquake.

2. Excessive Diaphragm Deflection—Because most of the floor diaphragms are constructed of finished wood flooring placed over ¾”-thick wood sheathing, they tend to be stiff compared with other types of wood diaphragms. This stiffness results in rotations about a vertical axis, accompanying translations in the direction of the open front walls of buildings, due to a lack of in-plane stiffness in these open fronts. Because there is little resistance in the masonry walls for out-of-plane loading, the walls allow large diaphragm displacements and cause the failure of the walls out of their plane. Large drifts occurring at the roof line can cause a masonry wall to overturn and collapse under its own weight.
3. Low Shear Resistance—The mortar used in these older buildings was often made of lime and sand, with little or no cement, and had very little shear strength. The bearing walls will be heavily damaged and collapse under large loads. (See Figure E-54)

Figure E-54 Damaged URM building, 1992 Big Bear earthquake.

4. Slender Walls —Some of these buildings have tall story heights and thin walls. This condition, especially in non-load-bearing walls, will result in buckling out-of-plane under severe lateral load. Failure of a non-load-bearing wall represents a falling hazard, whereas the collapse of a load-bearing wall will lead to partial or total collapse of the structure.

E.13.3 Common Rehabilitation Techniques

Over the last 10 years or more, jurisdictions in California have required that unreinforced masonry bearing-wall buildings be rehabilitated or demolished. To minimize the economical impact on owners of having to rehabilitate their buildings, many jurisdictions implemented phased programs such that the critical items were dealt with first. The following are the key elements included in a typical rehabilitation program.

1. Roof and floor diaphragms are connected to the walls for both anchorage forces (out of the plane of the wall) and shear forces (in the plane of the

wall). Anchorage connections are placed at 6 feet spacing or less, depending on the force requirements. Shear connections are usually placed at around 2 feet center to center. Anchors consist of bolts installed through the wall, with 6-inch-square washer plates, and connected to hardware attached to the wood framing. Shear connections usually are bolts embedded in the masonry walls in oversized holes filled with either a non-shrink grout or an epoxy adhesive. See Figure E-55.

2. In cases when the height to thickness ratio of the walls exceeds the limits of stability, rehabilitation consists of reducing the spans of the wall to a level that their thickness can support. Parapet rehabilitation consists of reducing the parapet to what is required for fire safety and then bracing from the top to the roof.
3. If the building has an open storefront in the first story, resulting in a soft story, part of the storefront is enclosed with new masonry or a steel frame is provided there, with new foundations.
4. Walls are rehabilitated by either closing openings with reinforced masonry or with reinforced gunite.

Figure E-55 Upper: Two existing anchors above three new wall anchors at floor line using decorative washer plates. Lower: Rehabilitation techniques include closely spaced anchors at floor and roof levels.

Appendix F

Earthquakes and How Buildings Resist Them

F.1 The Nature of Earthquakes

In a global sense, earthquakes result from motion between plates comprising the earth's crust (see Figure F-1). These plates are driven by the convective motion of the material in the earth's mantle between the core and the crust, which in turn is driven by heat generated at the earth's core. Just as in a heated pot of water, heat from the earth's core causes material to rise to the earth's surface. Forces between the rising material and the earth's crustal plates cause the plates to move. The resulting relative motions of the plates are associated with the generation of earthquakes. Where the plates spread apart, molten material fills the void. An example is the ridge on the ocean floor, at the middle of the Atlantic Ocean. This material quickly cools and, over millions of years, is driven by newer, viscous, fluid material across the ocean floor.

These large pieces of the earth's surface, termed tectonic plates, move very slowly and irregularly. Forces build up for decades, centuries, or millennia at the interfaces (or faults) between plates, until a large releasing movement suddenly occurs. This sudden, violent motion produces the nearby shaking that is felt as an earthquake. Strong shaking produces strong horizontal forces on structures, which can cause direct damage to buildings, bridges, and other man-made structures as well as triggering fires, landslides, road damage, tidal waves (tsunamis) and other damaging phenomena.

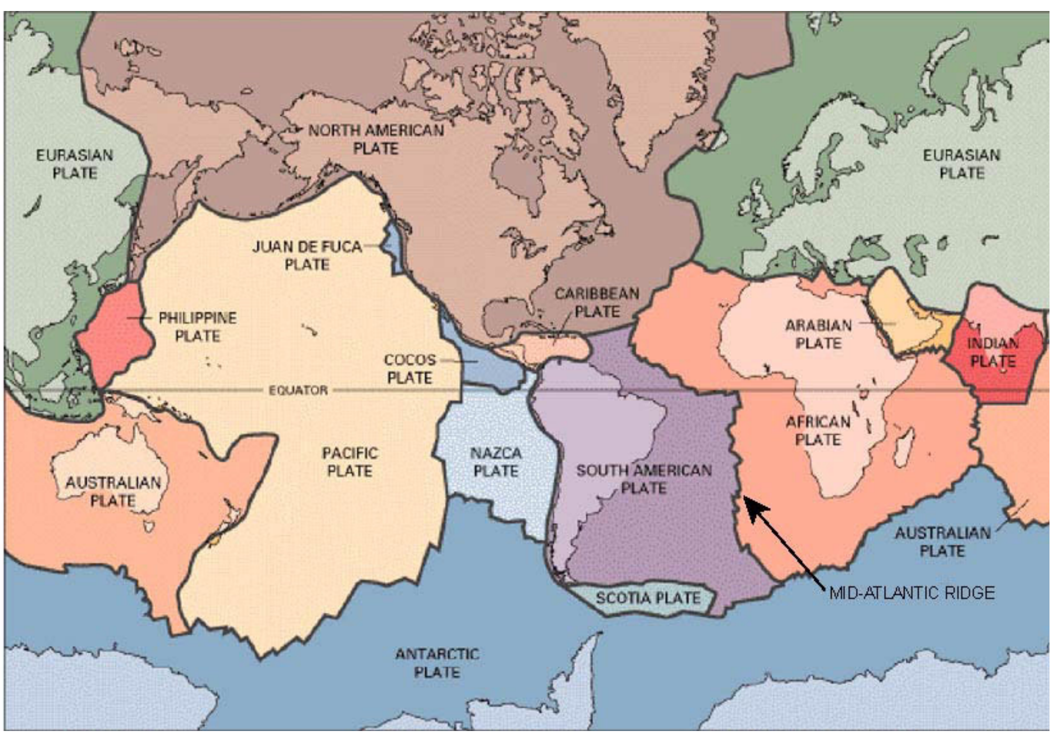


Figure F-1 The separate tectonic plates comprising the earth's crust superimposed on a map of the world.

A fault is like a "tear" in the earth's crust and its fault surface may be from one to over one hundred miles deep. In some cases, faults are the physical expression of the boundary between adjacent tectonic plates and thus are hundreds of miles long. In addition, there are shorter faults, parallel to, or branching out from, a main fault zone. Generally, the longer a fault, the larger magnitude earthquake it can generate. Beyond the main tectonic plates, there are many smaller sub-plates, "platelets" and simple blocks of crust which can move or shift due to the "jostling" of their neighbors and the major plates. The known existence of these many sub-plates implies that smaller but still damaging earthquakes are possible almost anywhere.

With the present understanding of the earthquake generating mechanism, the times, sizes and locations of earthquakes cannot be reliably predicted. Generally, earthquakes will be concentrated in the vicinity of faults, and certain faults are more likely than others to produce a large event, but the earthquake generating process is not understood well enough to predict the exact time of earthquake occurrence. Therefore, communities must be prepared for an earthquake to occur at any time.

Four major factors can affect the severity of ground shaking and thus potential damage at a site. These are the magnitude of the earthquake, the type of earthquake, the distance from the source of the earthquake to the site, and the hardness or softness of the rock or soil at the site. Larger earthquakes will shake longer and harder, and thus cause more damage. Experience has shown that the ground motion can be felt for several seconds to a minute or longer. In preparing for earthquakes, both horizontal (side to side) and vertical shaking must be considered.

There are many ways to describe the size and severity of an earthquake and associated ground shaking. Perhaps the most familiar are earthquake magnitude and Modified Mercalli Intensity (MMI, often simply termed "intensity"). Earthquake magnitude is technically known as the Richter magnitude, a numerical description of the maximum amplitude of ground movement measured by a seismograph (adjusted to a standard setting). On the Richter scale, the largest recorded earthquakes have had magnitudes of about 8.5. It is a logarithmic scale, and a unit increase in magnitude corresponds to a ten-fold increase in the adjusted ground displacement amplitude, and to approximately a thirty-fold increase in total potential strain energy released by the earthquake.

Modified Mercalli Intensity (MMI) is a subjective scale defining the level of shaking at specific sites on a scale of I to XII. (MMI is expressed in Roman numerals, to connote its approximate nature.) For example, slight shaking that causes few instances of fallen plaster or cracks in chimneys constitutes MMI VI. It is difficult to find a reliable precise relationship between magnitude, which is a description of the earthquake's total energy level, and intensity, which is a subjective description of the level of shaking of the earthquake at specific sites, because shaking intensity can vary with earthquake magnitude, soil type, and distance from the event.

The following analogy may be worth remembering: earthquake magnitude and intensity are similar to a light bulb and the light it emits. A particular light bulb has only one energy level, or wattage (e.g., 100 watts, analogous to an earthquake's magnitude). Near the light bulb, the light intensity is very bright (perhaps 100 foot-candles, analogous to MMI IX), while farther away the intensity decreases (e.g., 10 foot-candles, MMI V). A particular earthquake has only one magnitude value, whereas it has intensity values that differ throughout the surrounding land.

MMI is a subjective measure of seismic intensity at a site, and cannot be measured using a scientific instrument. Rather, MMI is estimated by scientists and engineers based on observations, such as the degree of disturbance to the ground, the degree of damage to typical buildings and the behavior of people. A more objective measure of seismic shaking at a site, which can be measured by instruments, is a simple structure's acceleration in response to the ground motion. In this *Handbook*, the level of ground shaking is described by the spectral response acceleration.

F.2 Seismicity of the United States

Maps showing the locations of earthquake epicenters over a specified time period are often used to characterize the seismicity of given regions. Figures F-2, F-3, and F-4 show the locations of earthquake epicenters[4] in the conterminous United States, Alaska, and Hawaii, respectively, recorded during the time period, 1977-1997. It is evident from Figures F-2 through F-4 that some parts of the country have experienced more earthquakes than others. The boundary between the North American and Pacific tectonic plates lies along the west coast of the United States and south of Alaska. The San Andreas fault in California and the Aleutian Trench off the coast of Alaska are part of this boundary. These active seismic zones have generated earthquakes with Richter

[4]An epicenter is defined as the point on the earth's surface beneath which the rupture process for a given earthquake commenced.

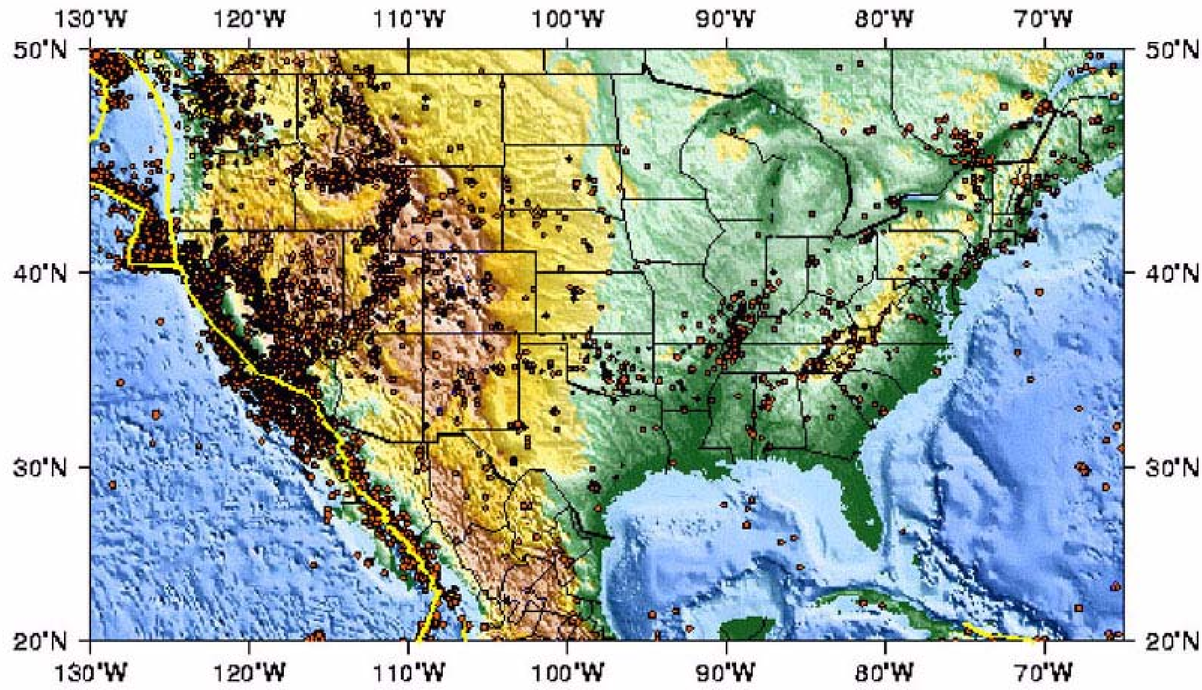


Figure F-2 Seismicity of the conterminous United States 1977 – 1997 (from the website at http://neic.usgs.gov/neis/general/seismicity/us.html). This reproduction shows earthquake locations without regard to magnitude or depth. The San Andreas fault and other plate boundaries are indicated with white lines.

magnitudes greater than 8. There are many other smaller fault zones throughout the western United States that are also participating intermittently in releasing the stresses and strains that are built up as the tectonic plates try to move past one another. Because earthquakes always occur along faults, the seismic hazard will be greater for those population centers close to active fault zones.

In California the earthquake hazard is so significant that special study zones have been created by the legislature, and named Alquist-Priola Special Study Zones. These zones cover the larger known faults and require special geotechnical studies to be performed in order to establish design parameters.

On the east coast of the United States, the sources of earthquakes are less understood. There is no plate boundary and few locations of faults are known. Therefore, it is difficult to make statements about where earthquakes are most likely to occur. Several significant historical earthquakes have occurred, such as in Charleston, South Carolina, in 1886 and New Madrid, Missouri, in 1811 and 1812, indicating that there is potential for large earthquakes. However, most earthquakes in the eastern United States are smaller magnitude events. Because of regional geologic differences, specifically, the hardness of the crustal rock, eastern and central U.S. earthquakes are felt at much greater distances from their sources than those in the western United States, sometimes at distances up to a thousand miles.

F.3 Earthquake Effects

Many different types of damage can occur in buildings. Damage can be divided into two categories: structural and nonstructural, both of which can be hazardous to building occupants. Structural damage means degradation of the building's structural support systems (i.e., vertical- and lateral-force-resisting systems), such as the building frames and walls. Nonstructural damage refers to any damage that does not affect the integrity of the structural support systems. Examples of nonstructural damage are chimneys collapsing, windows breaking, or ceilings falling. The type of damage to be expected is a complex issue that depends on the structural type and age of the building, its configuration, construction materials, the site conditions, the proximity of the building to neighboring buildings, and the type of nonstructural elements.

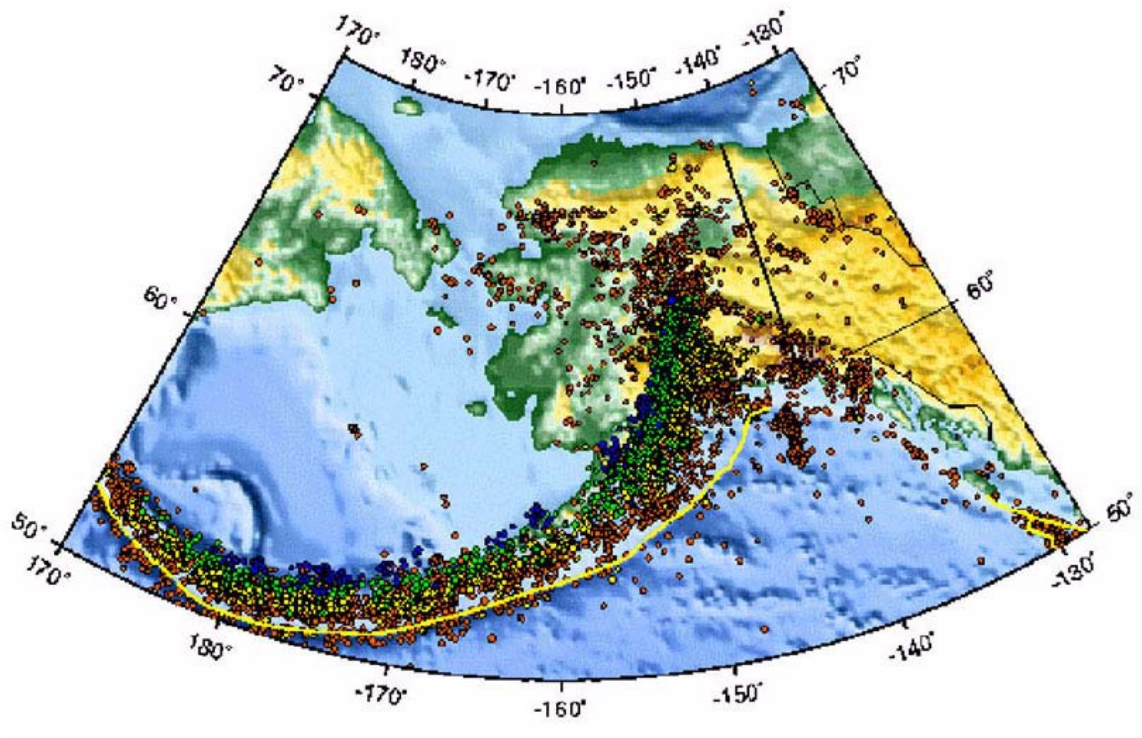


Figure F-3 Seismicity of Alaska 1977 – 1997. The white line close to most of the earthquakes is the plate boundary, on the ocean floor, between the Pacific and North America plates.

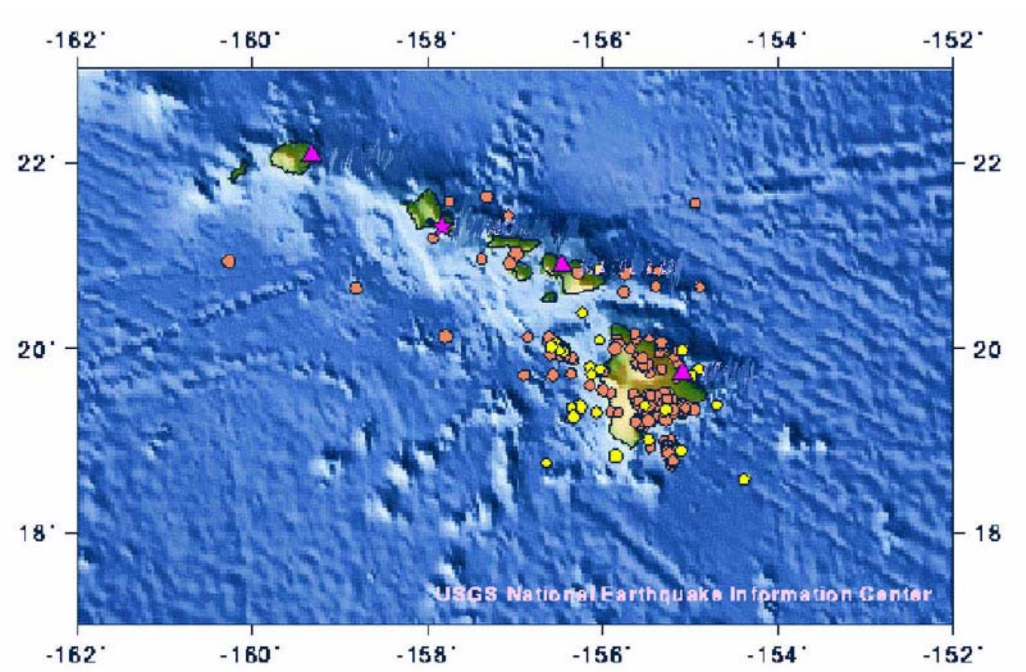


Figure F-4 Seismicity of Hawaii 1977 – 1997. See Figure F-2 caption.

When strong earthquake shaking occurs, a building is thrown mostly from side to side, and also up and down. That is, while the ground is violently moving from side to side, taking the building foundation with it, the building structure tends to stay at rest, similar to a passenger standing on a bus that accelerates quickly. Once the building starts moving, it tends to continue in the same direction, but the ground moves back in the opposite direction (as if the bus driver first accelerated quickly, then suddenly braked). Thus the building gets thrown back and forth by the motion of the ground, with some parts of the building lagging behind the foundation movement, and then moving in the opposite direction. The force F that an upper floor level or roof level of the building should successfully resist is related to its mass m and its acceleration a, according to Newton's law, $F = ma$. The heavier the building the more the force is exerted. Therefore, a tall, heavy, reinforced-concrete building will be subject to more force than a lightweight, one-story, wood-frame house, given the same acceleration.

Damage can be due either to structural members (beams and columns) being overloaded or differential movements between different parts of the structure. If the structure is sufficiently strong to resist these forces or differential movements, little damage will result. If the structure cannot resist these forces or differential movements, structural members will be damaged, and collapse may occur.

Building damage is related to the duration and the severity of the ground shaking. Larger earthquakes tend to shake longer and harder and therefore cause more damage to structures. Earthquakes with Richter magnitudes less than 5 rarely cause significant damage to buildings, since acceleration levels (except when the site is on the fault) and duration of shaking for these earthquakes are relatively small.

In addition to damage caused by ground shaking, damage can be caused by buildings pounding against one another, ground failure that causes the degradation of the building foundation, landslides, fires and tidal waves (tsunamis). Most of these "indirect" forms of damage are not addressed in this *Handbook*.

Generally, the farther from the source of an earthquake, the less severe the motion. The rate at which motion decreases with distance is a function of the regional geology, inherent characteristics and details of the earthquake, and its source location. The underlying geology of the site can also have a significant effect on the amplitude of the ground motion there. Soft, loose soils tend to amplify the ground motion and in many cases a resonance effect can make it last longer. In such circumstances, building damage can be accentuated. In the San Francisco earthquake of 1906, damage was greater in the areas where buildings were constructed on loose, man-made fill and less at the tops of the rocky hills. Even more dramatic was the 1985 Mexico City earthquake. This earthquake occurred 250 miles from the city, but very soft soils beneath the city amplified the ground shaking enough to cause weak mid-rise buildings to collapse (see Figure F-5). Resonance of the building frequency with the amplified ground shaking frequency played a significant role. Sites with rock close to or at the surface will be less likely to amplify motion. The type of motion felt also changes with distance from the earthquake. Close to the source the motion tends to be violent rapid shaking, whereas farther away the motion is normally more of a swaying nature. Buildings will respond differently to the rapid shaking than to the swaying motion.

Each building has its own vibrational characteristics that depend on building height and structural type. Similarly, each earthquake has its own vibrational characteristics that depend on the geology of the site, distance from the source, and the type and site of the earthquake source mechanism. Sometimes a natural resonant frequency of the building and a prominent frequency of the earthquake motion are similar and cause a sympathetic response, termed resonance. This causes an increase in the amplitude of the building's vibration and consequently increases the potential for damage.

Resonance was a major problem in the 1985 Mexico City earthquake, in which the total collapse of many mid-rise buildings (Figure F-5) caused many fatalities. Tall buildings at large distances from the earthquake source have a small, but finite, probability of being subjected to ground motions containing frequencies that can cause resonance.

Where taller, more flexible, buildings are susceptible to distant earthquakes (swaying motion) shorter

Figure F-5 Mid-rise building collapse, 1985 Mexico City earthquake.

Figure F-6 Near-field effects, 1992 Landers earthquake, showing house (white arrow) close to surface faulting (black arrow); the insert shows a house interior.

and stiffer buildings are more susceptible to nearby earthquakes (rapid shaking). Figure F-6 shows the effects on shorter, stiffer structures that are close to the source. The inset picture shows the interior of the house. Accompanying the near field effects is surface faulting also shown in Figure F-6.

The level of damage that results from a major earthquake depends on how well a building has been designed and constructed. The exact type of damage cannot be predicted because no two buildings undergo identical motion. However, there are some general trends that have been observed in many earthquakes.

- Newer buildings generally sustain less damage than older buildings designed to earlier codes.
- Common problems in wood-frame construction are the collapse of unreinforced chimneys (Figure F-7) houses sliding off their foundations (Figure F-8),collapse of cripple walls (Figure F-9), or collapse of post and pier foundations (Figure F-10). Although such damage may be costly to repair, it is not usually life threatening.
- The collapse of load bearing walls that support an entire structure is a common form of damage in unreinforced masonry structures (Figure F-11).

Figure F-7 Collapsed chimney with damaged roof, 1987 Whittier Narrows earthquake.

- Similar types of damage have occurred in many older tilt-up buildings (Figure F-12).

From a life-safety perspective, vulnerable buildings need to be clearly identified, and then strengthened or demolished.

F.4 How Buildings Resist Earthquakes

As described above, buildings experience horizontal distortion when subjected to earthquake motion. When these distortions get large, the damage can be catastrophic. Therefore, most buildings are designed

Figure F-8 House that slid off foundation, 1994 Northridge earthquake.

Figure F-9 Collapsed cripple stud walls dropped this house to the ground, 1992 Landers and Big Bear earthquakes.

Figure F-10 This house has settled to the ground due to collapse of its post and pier foundation.

Figure F-11 Collapse of unreinforced masonry bearing wall, 1933 Long Beach earthquake.

Figure F-12 Collapse of a tilt-up bearing wall.

with lateral-force-resisting systems (or seismic systems), to resist the effects of earthquake forces. In many cases seismic systems make a building stiffer against horizontal forces, and thus minimize the amount of relative lateral movement and consequently the damage. Seismic systems are usually designed to resist only forces that result from horizontal ground motion, as distinct from vertical ground motion.

The combined action of seismic systems along the width and length of a building can typically resist earthquake motion from any direction. Seismic systems differ from building to building because the type of system is controlled to some extent by the basic layout and structural elements of the building. Basically, seismic systems consist of axial-, shear- and bending-resistant elements.

In wood-frame, stud-wall buildings, plywood siding is typically used to prevent excessive lateral deflection in the plane of the wall. Without the extra strength provided by the plywood, walls would distort excessively or “rack,” resulting in broken windows and stuck doors. In older wood frame houses, this resistance to lateral loads is provided by either wood or steel diagonal bracing.

The earthquake-resisting systems in modern steel buildings take many forms. In moment-resisting steel frames, the connections between the beams and the columns are designed to resist the rotation of the column relative to the beam. Thus, the beam and the column work together and resist lateral movement and lateral displacement by bending. Steel frames sometimes include diagonal bracing configurations, such as single diagonal braces, cross-bracing and "K-bracing." In braced frames, horizontal loads are resisted through tension and compression forces in the braces with resulting changed forces in the beams and columns. Steel buildings are sometimes constructed with moment-resistant frames in one direction and braced frames in the other.

In concrete structures, shear walls are sometimes used to provide lateral resistance in the plane of the wall, in addition to moment-resisting frames. Ideally, these shear walls are continuous reinforced-concrete walls extending from the foundation to the roof of the building. They can be exterior walls or interior walls. They are interconnected with the rest of the concrete frame, and thus resist the horizontal motion of one floor relative to another. Shear walls can also be constructed of reinforced masonry, using bricks or concrete blocks.